

SHOTCRETE: ELEMENTS OF A SYSTEM

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Shotcrete: Elements of a System

Editor

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Preface

The Third International Conference on Engineering Developments in Shotcrete has been organized by the Australian Shotcrete Society to encourage discussion about on-going development of shotcrete technology within the construction and mining industries, and disseminate recent information concerning advances in this field. It was held in Queenstown, New Zealand, with the financial support of companies involved in the shotcrete industry both within the Australia/New Zealand area, and the wider world. Planning and preparation for the conference was strongly guided by the organizing committee within the Australian Shotcrete Society, particularly Tony Cooper and John Brown. The American Shotcrete Association also assisted in promoting the conference, for which the society is indebted.

Use of shotcrete for ground support and other applications continues to increase internationally. The reasons for this include an increase in awareness among engineers and contractors, particularly in developing countries, of the possibilities and economies that this process of concrete placement offers. The underground mining industry in Australia is at the forefront of implementation of new shotcrete technologies, with almost universal adoption of Fibre Reinforced Shotcrete for ground support. For this reason many of the papers included in this volume are based on practices within this industry. The more conservative attitudes prevalent in other countries, and in the civil tunneling industry in general, have resulted in a slower rate of uptake of recent innovations. Some of these innovations were reported at the first and second conferences in this series, held in Hobart in 2001, and Cairns in 2004, as well as at the two long standing series of conferences on shotcrete held by the Norwegian Concrete Association and Engineering Conferences International. Developments and innovations related to shotcrete have arisen in many parts of the world in recent years and this is evident in the diversity of authors represented at this conference.

Each paper presented herein was independently and anonymously reviewed by up to three peers from within the industry. The process of review was lengthy and rigorous, and I would like to extend my thanks to the reviewers for their assistance in making this conference possible. I would also like to thank the sponsors for providing such generous support and thereby contributing to a rewarding conference experience for the delegates.

E.S. Bernard
Conference Chairman and Editor

Acknowledgements

A successful conference requires the involvement of a large number of people. The organizing committee would like to thank the following individuals and companies for their support in making this conference possible. In particular, the committee would like to thank Vickie Hill, of Queenstown Destination Management, for the day-to-day preparations leading up to the conference and organization of delegate registration within Queenstown.

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A comparison of models for shotcrete in dynamically loaded rock tunnels

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ABSTRACT: During blasting in tunnels and mines, the shotcrete-rock interaction is influenced by propagating stress waves. Shotcrete support in hard rock tunnels is here studied through numerical analysis and comparisons with previous numerical results, measurements and observations in situ. The stress response in the shotcrete closest to the rock when exposed to P-waves striking perpendicularly to the shotcrete-rock interface is simulated. The first model tested is an elastic stress wave model, which is one-dimensional with the shotcrete assumed linearly elastic. The second is a structural dynamic model that consists of masses and spring elements. The third model is a finite element model implemented using the Abaqus/Explicit program. Two methods are used for the application of incident disturbing stress waves: as boundary conditions and as inertia loads. Results from these three types of models are compared and evaluated as a first step before a future extension to more detailed analyses using 3D models.

1 INTRODUCTION

The main design principle for rock support is to help the rock carry its inherent loads. The rock support is generally designed for static loading conditions but in many cases, however, the openings are also subjected to dynamic loads. One example is rock bursts that can cause serious damage to underground structures. Another source of dynamic loads is detonation of explosives during excavations of tunnels and underground spaces. These detonations give rise to stress waves that transport energy through the rock and these may, depending on the magnitude of the waves, cause severe damage to permanent installations and support systems within the rock, such as shotcrete.

In tunnelling, the search for a more time-efficient construction process naturally focuses on the possibilities of reducing the time periods of waiting between stages of construction. As an example, the driving of two parallel tunnels requires coordination between the two excavations so that blasting in one tunnel does not, through vibrations, damage temporary support systems in the other tunnel prior to installation of a sturdier, permanent support. There also arise similar problems in mining. To be able to excavate as much ore volume as possible, the grid of drifts in a modern mine is dense. This means that supporting systems in one drift are likely to be affected by vibrations in a neighbouring drift.

This research project was initiated to study how close in time and distance to shotcrete safe blasting

can take place. It involves development of sophisticated dynamic finite element (FE) models which will be evaluated and refined through comparisons between calculated and measured data. At first, existing simple prototype models will be evaluated through calculations and comparisons with existing data. The models will then be further refined and re-modelled using FE programs and in the following step the prototype models will be replaced by a model based on solid FE elements. Non-linear material properties will be introduced, replacing the initial, elastic approach. The model will be used to study real in situ cases with realistic and complex geometry, such as shotcrete on tunnel walls and ceilings behind a tunnel front where blasting takes place. The results will lead to the establishment of recommendations and guidelines for practical use in e.g. civil engineering underground work, tunnelling and mining. This paper presents preliminary results, ongoing and future research within the project, assuming fully hardened shotcrete.

2 SHOTCRETE AND BLASTING

There are few published reports and papers on tests conducted in tunnels and mines where shotcrete has been subjected to vibrations from large-scale blasting. Due to the lack of knowledge, unnecessarily strict guidelines for acceptable vibration levels from blasting close to hardened as well as newly sprayed shotcrete are often used. For safe underground construction work, this leads to longer production times

and higher costs. The guidelines often contain allowed vibration velocities, expressed as peak particle velocities (ppv), which are difficult to translate into minimum distance and shotcrete age. In the following a short description of stress waves in rock is given, followed by a summary of earlier interesting experiences and research on vibration resistance of shotcrete.

2.1 Stress waves in rock

Detonations in rock give rise to stress waves that transport energy through the rock, towards possible free rock surfaces. Wave motion can be described as movement of energy through a material, transportation of energy achieved by particles translating and returning to equilibrium after the wave has passed (Bodare 1997). The propagation velocity is governed by the type of rock and is different for different types of waves, such as compression waves (P-waves) and shear waves (S-waves). The P-wave propagates faster than the S-wave and is therefore the first to reach an observation point when both wave types have been generated simultaneously at a distant source, e.g. an earthquake or a blast round. There are also other types such as Rayleigh waves that appear on surfaces, see e.g. Dowding (1996).

As each wave passes, the motion of the particles in the rock can be described in three dimensions, either as displacements, velocities or accelerations. When a wave-front reflects at a free surface, such as that of a tunnel, the particle velocities are doubled and the stresses are zero over the surface. This means that a compressive wave reflects backwards as a tensile wave, etc. Shotcrete sprayed on rock exposed to blasting will thus be affected by incoming stress waves that reflect at the shotcrete-rock interface. When exposed to incoming stress waves, the inertia forces caused by the accelerations acting on the shotcrete give rise to stresses at the shotcrete-rock interface which may cause adhesive failure. It is also possible that the shotcrete may fail due to low tensile strength. The particle velocities that can be measured remote from a detonation in rock will show a decrease in magnitude with increasing distance to the source of explosion. This decay is caused by geometrical spreading and hysteretic damping in the rock (Dowding 1996) and is governed by a relation of the form

$$v_{\max} = a_1 \left(\frac{R}{\sqrt{Q}} \right)^{-\beta} \quad (1)$$

where $v_{\max}$ is the peak particle velocity (ppv) at a distance R from the point charge with the weight

of the explosives Q and the constants a_1 and β . Equation (1) is valid only for situations where R is large compared to the length of the explosive charge, thus assuming a concentration of the explosive charges. As previously mentioned $v_{\max}$ of an incoming wave will be doubled when reflected at the free surface of, for example, a tunnel, and therefore must be obtained from Equation (1) through multiplication by two (Ansell 1999).

The time-dependent stress from a propagating longitudinal wave is

$$\sigma(t) = \rho_{\text{rock}} c_{\text{rock}} v(t) \quad (2)$$

where ρ_{rock} is the rock density and $v(t)$ the particle velocity. The wave propagation velocity in elastic materials is

$$c_{\text{rock}} = \sqrt{\frac{E_{\text{rock}}}{\rho_{\text{rock}}}} \quad (3)$$

where E_{rock} is the elastic modulus of the rock material. It should be noted that the allowable dynamic load on a structural system also depends on the frequency content, ie. higher ppv levels can be accepted for high frequencies. The highest frequencies are however filtered out as the waves propagate through the rock.

2.2 Dynamic load capacity

A couple of interesting reports present in situ tests conducted in tunnels and mines where fully hardened shotcrete on rock have been subjected to blast induced vibrations. Kendorski et al. (1973) carried out in situ tests to determine how a shotcrete lining was affected by standard drift blasts at various distances from the lining. A standard blast that consisted of 409 kg premixed ammonium and fuel oil (ANFO) showed that there was no bond failure at the shotcrete-rock interface. The tests revealed that cracks started to appear in the shotcrete when the detonations occurred at a distance of 16.5 m and that the function of the lining was considerably reduced when detonating from 12.2 m. No vibration levels were recorded during these tests. McCreath et al. (1994), Tannant & McDowell (1993), and Wood & Tannant (1994) presented results from tests carried out in a Canadian gold-mine where steel fibre-reinforced and steel mesh-reinforced shotcrete linings were subjected to vibrations from explosions. During the tests, it was found that steel fibre-reinforced shotcrete can maintain its functionality even though exposed to vibration levels of 1500–2000 mm/s. It was seen that mesh-reinforced shotcrete performs better

than steel fibre-reinforced shotcrete under very severe dynamic loading conditions. This is due to its ability to retain broken rock even when extensively cracked, which is not the case with fibre-reinforced shotcrete. It is reported that the shotcrete linings were partially cracked and that no shotcrete slabs were displaced or ejected by the blasting, nor was any significant increase in drumminess found by manual hammer sounding as the blasting continued. The latter indicates that the shotcrete-rock adhesive bond was undamaged. Further, it is suggested that mesh-reinforced shotcrete may remain partially functional when subjected to particle velocities as high as 2000–6000 mm/s (McCreath et al. 1994). These conclusions were based on empirical observations by the ground control staff at operating mines in the Sudbury area (Ontario, Canada).

The function of shotcrete in the support of burst prone ground has further been investigated through static and dynamic testing of shotcrete panels by Beauchamp (1995), also presented by Tannant et al. (1995, 1996). In their test program, impacts of rock blocks ejected from medium-sized rock bursts were simulated by using weights dropped on shotcrete panels. Different loading rates and panel configurations were tested. The result was

presented as a relationship between panel damage, energy dissipation and displacement.

3 PRELIMINARY RESULTS

As a continuation to the research described in the previous section in situ tests with small and larger scale detonations were conducted, followed by numerical modelling (Ansell 1999, 2004a, 2004b, 2005, 2007). These results are briefly summarized and commented on in the following.

3.1 *In situ tests*

Tests on young shotcrete were performed on site in the Kiirunavaara iron-ore mine, situated in the north of Sweden (Ansell 1999, 2004a). The test sites were situated on both sides of a tunnel that was not originally reinforced by shotcrete. A total of four tests were carried out, each performed with a unique type of explosive charge not repeated in any of the other tests. Two shotcrete areas of varying age, one young and one very young, were subjected to vibrations in each test and no shotcrete area was subjected to vibrations more than once. The geometry of a test site is described in Figure 1.

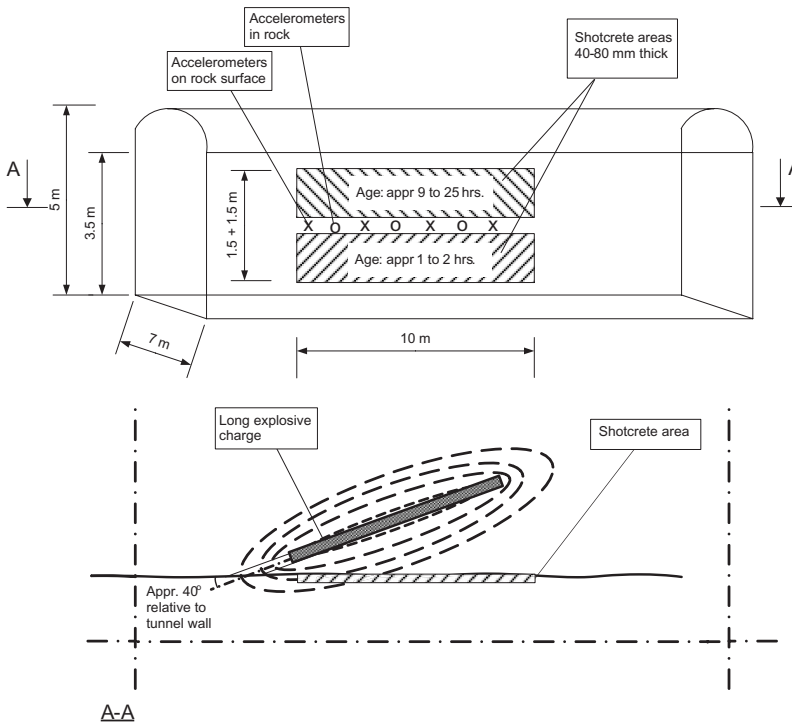


Figure 1. Schematic view of a test site. Explosive charge in rock behind shotcrete areas (Ansell & Holmgren 2001).

Measurement points for placement of accelerometers were located in the same horizontal plane as the charge holes, four points placed on the tunnel surface and three points 0.5 m into the rock. Three plastic pipes for mounting of accelerometers inside the rock were applied through injection by cement grout and four aluminium plates for accelerometers were bolted to the rock surface.

At each measurement point two accelerometers were mounted, normal to and parallel with the tunnel walls, i.e. accelerations were recorded using a total of 14 channels in each test. In the evaluation model used, it was assumed that only the first peak in each record provided a correct maximum velocity, i.e. that no reflections and superpositions had at such an early stage distorted the signals.

This method of determining the maximum particle velocity, or peak particle velocity (ppv), is sometimes referred to as the “first-arrival principle”. In the ideal case, the ppv will coincide with the point where both of the two measured (in reality, three) acceleration components pass through the zero level simultaneously after the first half period of vibration. An example of a measured acceleration signal is shown in Figure 2. The equation

$$v_{\max} = 700 \left(\frac{R}{\sqrt{Q}} \right)^{-1.70} \quad [\text{mm/s}] \quad (4)$$

was derived through regression analysis (Ansell 1999) from all the numerically determined ppv values and is valid for estimation of the vibration velocities inside the rock at Kiirunavaara.

3.2 Numerical modelling

A first step in modelling of shotcrete exposed to vibrations was taken by Ansell (1999). The in situ tests with young shotcrete were followed by numerical modelling with a first approach based

on elastic stress wave theory. This resulted in a model for one-dimensional analysis of shotcrete on rock through which elastic stress waves propagate towards the shotcrete. Acceleration–time histories recorded in situ were used as loads. In a following project, an elastic finite element model was proposed (Ansell 2004b, 2005). In contrast to the earlier one-dimensional stress wave model, this finite element model is two-dimensional. This facilitates the calculation of a two-dimensional displacement field instead of the displacement at an isolated node. The model consists of beam elements that are used for representation of the flexural stiffness and mass of the shotcrete and the fractured rock closest to the rock surface. Spring elements are used to obtain elastic coupling between shotcrete and rock. The modelling approach is similar to that of a building during an earthquake, with accelerations measured in situ used as loads.

3.3 Recommendations

The preliminary numerical results have been verified through comparisons with observations and results in situ, from full scale tests (Ansell 2004a) and from mining operations (Ansell & Malmgren 2003 and Ansell 2007). The results showed that the main failure mode is loss of the adhesive bond between shotcrete and rock. The work resulted in recommendations of peak particle velocities (ppv) calculated from the scaling relation given by Equation (4). These are here given in Table 1, also including a summary of the results referred to in section 2.2.

Table 1. Vibration peak particle velocities (ppv) when shotcrete damage occurs. Values from observations, measurements and calculations (Ansell 2007).

ppv at damage (mm/s)		Comments
From measurements:		
Kiirunavaara tests (Ansell 1999, 2004a)	500–1000	Young shotcrete
Japanese tunnelling (Nakano et al. 1993)	700–1450	
Mining, full scale (Kendorski 1973)	1000–1800	Estimated
Canadian tests (Wood & Tannant 1994, McCreath et al. 1994)	1500–2000	Steel fibre-reinforced
Recommendation: (Ansell 2005, 2007)		
	2300–3000	25 mm shotcrete
	700–710	50 mm shotcrete
	230–250	100 mm shotcrete

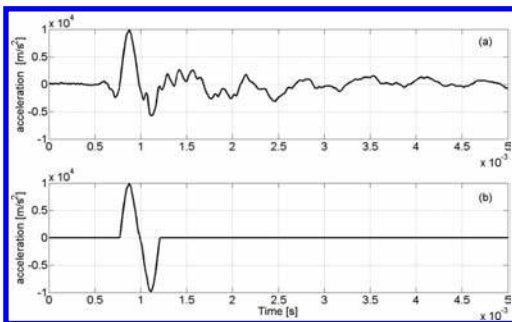


Figure 2. Example of measured accelerations (a) and part of the signal used for numeric analysis (b). Direction of measurement is normal to the rock surface.

4 NUMERICAL MODELS

The evaluated models are one-dimensional and the point of detonation was assumed to be positioned within the rock, perpendicular to the point of observation on the free surface. Hence, all the models describe the stress response at one discrete point. The incident disturbing stress wave used in the comparison of the different types of models was derived from the scaling law for rock, Equation (1), together with the presumption of a characteristic frequency. That is, the applied disturbance is a theoretically defined approximation to those observed in situ. It thus provides a clear relation between the disturbance expressed as acceleration, velocity, displacement and stress and therefore enables comprehensible comparisons to be made between the different types of models and methods for defining the incident disturbing stress wave. The evaluation was made for a detonation of 2 kg of explosives concentrated at one point positioned 4 m below the rock surface. The shotcrete thickness was kept constant at 0.05 m.

4.1 Material properties

A simplification that was made throughout this study was that both the rock and the shotcrete were considered to behave in a strictly elastic manner. That is, no plastic deformations or permanent failure, e.g. crushing, cracking or debonding, were considered within the models and a linear elastic relationship between stress and strain was assumed.

It is difficult to define material parameters for a fractured rock mass since its behaviour is site-specific and this affects the characteristics of propagating stress waves (Wersäll 2008). Here, a Young's modulus of elasticity of 40 GPa was chosen for the rock material, assumed to be linearly elastic. Furthermore, the rock properties in the numerical examples were chosen to represent typical Swedish granite; a density of 2400 kg/m³, a compressive strength of 160–240 MPa and a Poisson's ratio set to zero, since the problem treated here was purely one-dimensional. The longitudinal wave velocity was 4000 m/s and the particle velocity was assumed to follow the relation given in Equation (4).

A shotcrete lining can be either un-reinforced or reinforced, for instance with steel fibres, and its strength will be dependent on the composition of the shotcrete, the age of the maturing shotcrete and its ability to adhere to the rock surface. The adhesion strength perpendicular to the rock surface often lies within 0 and 2 MPa (Ansell 1999). Since this strength often is lower than both the compressive and the tensile strengths the adhesive bond to the rock will be crucial. In the numerical examples

fully hardened (28 days) un-reinforced shotcrete was considered and the adhesion strength towards granite should for comparisons with older results be set to approximately 0.5 MPa. The shotcrete was furthermore assumed to have a density of 2100 kg/m³ and a modulus of elasticity of 30 GPa. The chosen material properties were similar to that of the shotcrete used by LKAB in the Kiirunavaara mine, see Ansell (1999).

4.2 Elastic stress wave model

In elastic analysis, the material is often assumed to be infinite (or semi-infinite), linear elastic, continuous and isotropic. Rock mass is usually none of the above but these assumptions can be valid for deep tunnels subjected to small strains, e.g. from blast loads. An elastic stress wave model for analyses of shotcrete on vibrating rock, based on these assumptions, has been tested by James (1998) and Ansell (1999). The model was implemented with the numeric software Matlab (Mathworks 2009). Since the longitudinal P-wave was presumed to cause the first and major stress influence, this type of wave was the only one considered in the model. The effect of damping was disregarded and the stresses were assumed to follow Equation (2).

A graphical description of the model is shown in Figure 3 where the shotcrete lining is divided into n elements, the thickness of each element, Δx , being determined so that in each time increment the wave has propagated a distance of one element. The stress wave was assumed to be transmitted and reflected at boundaries and subsequently the stresses travelling in the positive and negative directions were superimposed to give the total stress within each element of the shotcrete lining.

4.3 Structural dynamic model

This mass-spring model includes fractured rock and the shotcrete lining, as lumped masses positioned

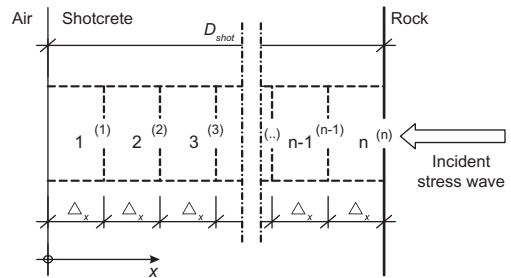


Figure 3. Elastic stress wave model of a shotcrete lining with thickness D_{shot} applied to a rock surface and exposed to a stress wave. The shotcrete lining is divided into n elements, each with thickness Δx (Nilsson 2009).

at the centres of gravity of each segment of the model, as seen in Figure 4. These were connected to the adjacent masses with springs corresponding to the axial stiffness of the shotcrete or fractured rock element. The degree of discretisation of the shotcrete has almost no influence on the maximum stress level, which developed within the shotcrete for both models. The greatest stress level is obtained in the shotcrete closest to the rock, and thereafter the stress level descended to zero towards the free surface. Since the total number of elements required is small for a model with masses at the centre of gravity, compared to if the masses are positioned at the interfaces (Nilsson 2009), this type of model was considered here. For the numerical analyses presented the shotcrete was modelled with one element, whilst the rock was divided into 40 elements. Also in this case Matlab (Mathworks 2009) was used for the analyses.

4.4 Finite element model

The engineering simulation software Abaqus (Simulia 2009a, 2009b) was used to create finite element models analyzed with the Abaqus/Explicit solver. A condition of plane strain was presumed in the directions parallel to the free surface of shotcrete. This implies that three-dimensional solid elements can be used to model the rock and shotcrete. Furthermore, when modelling a rectangular rod of rock and shotcrete with an incident longitudinal wave striking the inner surface of rock (or shotcrete) three-dimensional solid elements can be used as well. Simplifying the model to a rectangular rod of rock and shotcrete also means that the displacements in the directions transverse to the wave propagation can be constrained using displacement boundary conditions, thus restricting

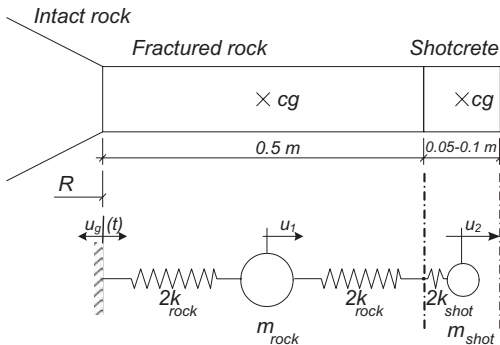


Figure 4. Structural dynamic model including both fractured rock and shotcrete exposed to ground motion, with masses at the centre of gravity of each element. R denotes the distance used in the scaling law for rock and u_i the displacements normal to the rock surface.

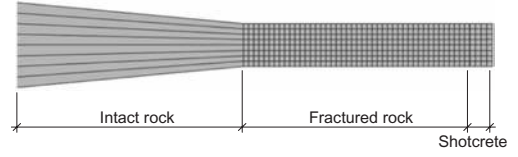


Figure 5. An Abaqus-model with intact rock (infinite elements), fractured rock (finite elements) and shotcrete (finite elements) modelled with plane strain elements. Displacements in the vertical direction are restricted (Nilsson 2009).

Table 2. Properties for Abaqus/Explicit model consisting of shotcrete lining and fractured rock.

Specific properties	Part		
	Shotcrete	Fractured rock	Intact rock
Cross section	$0.1 \times 0.1 \text{ m}^2$	$0.1 \times 0.1 \text{ m}^2$	—
Thickness	0.05 m	0.50 m	—
Elements	8×4	8×40	8
Element type	C3D8R*	C3D8R*	infinite

* C3D8R Explicit, continuum, 3D stress, 8-node linear, reduced integration, hourglass control.

the model to primarily describe particle displacements in the wave propagation direction.

Figure 5 shows the tested Abaqus-model with plane strain elements and features according to Table 2. The fractured rock and the shotcrete were modelled with finite elements and the intact rock with infinite elements. Tie constraints between rock and shotcrete were used and the infinite elements simulate a boundary with no reflection, also called a quiet boundary. The element size within the model was chosen to be 0.0125 m (Nilsson 2009). The incident disturbing stress wave, caused by the explosion within the rock, was applied in two principally different ways; as a boundary condition, i.e. as acceleration, velocity or displacement, and as a stress-load.

It is to be noted that the amplitudes describing the stress wave are dependent on the distance R to the point of detonation.

5 RESULTS

Results using theoretically determined (forecasted) particle accelerations and in situ measured accelerations as loads will be shown. First are the stress responses from all three types of models—elastic stress wave model, structural dynamic model and finite element model—using forecasted particle accelerations presented together with a comparison

of the results. Thereafter, the elastic stress wave model is used to simulate the stress responses from two acceleration-time histories, measured in situ.

5.1 Model comparisons

The incident disturbing waves used as loads in this comparison were assumed sinusoidal acceleration variations with peak particle acceleration of either 2259 m/s^2 at 0.5 m below the rock surface or 1849 m/s^2 right under the rock surface, respectively. A frequency of 2000 Hz was considered, see also Nilsson (2009). In the examples of the elastic stress wave model and the Abaqus/Explicit models the stress responses were obtained by using an initial velocity conditions equal to zero, as well as an initial velocity condition equal to the maximum amplitude, as in Figure 6. The incident stress variation was obtained using Equation (2). When boundary conditions were used to describe the incident disturbance the stresses were set to zero after passing of the wave, to prevent the particle (node) acceleration at the interface between the intact and the fractured rock increasing at an unreasonable rate.

The stress responses at the shotcrete element closest to the rock surface are plotted for all models in Figures 7 and 8, with the incident disturbance defined at the interface between intact and fractured rock and between rock and shotcrete, respectively. In this case, the initial velocity condition was set to zero, as shown in Figure 6(b).

The case described in Figure 6(a) corresponds to a suddenly applied stress level followed by a cosine-period. This results in stress peaks that propagate through the elements of the elastic stress wave model and the Abaqus/Explicit finite element model. The stress responses in the shotcrete element closest to the rock surface when an incident disturbance is defined at the interface between intact and fractured rock and between rock and shotcrete, respectively, are as shown in Figures 9 and 10.

5.2 From in situ measurements

As part of the testing and evaluation of the models, in situ recorded acceleration-time history data from the Kiirunavaara mine (Ansell 1999) is also used as input. For the numerical examples, the accelerometer readings shown in Figure 2 were chosen. The sampling frequency f_{samp} of these in situ measured accelerations is 100 kHz , giving a time interval Δt of 10^{-5} s . For an adequate description of the sine-wave accelerations in the previous examples, a time increment Δt of 10^{-6} s was needed and therefore the same time interval was used also for the in situ measured accelerations. Thus, a linear interpolation

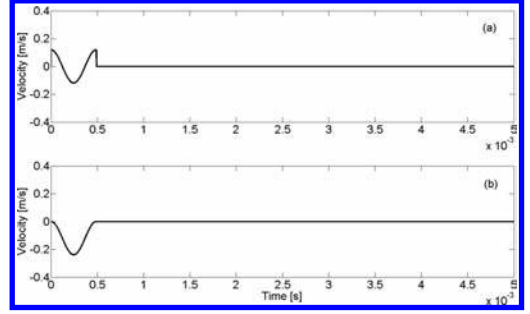


Figure 6. Particle velocity derived from accelerations, with the condition of initial velocity equal to (a) its maximum amplitude and (b) to zero.

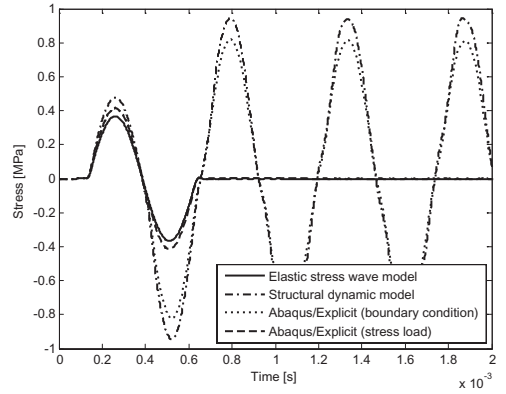


Figure 7. Stress responses in the shotcrete element closest to the rock for models with the incident disturbance defined at the interface between intact and fractured rock for all models but the elastic stress wave model. The initial velocity condition was set to zero (Nilsson 2009).

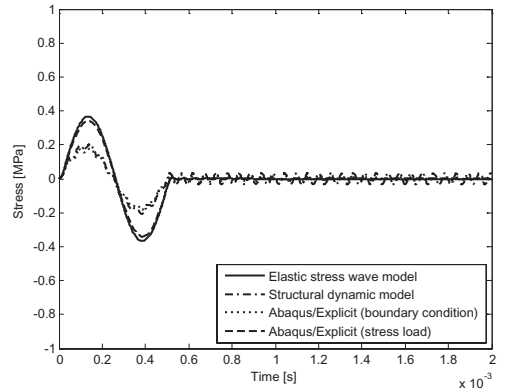


Figure 8. Stress response in the shotcrete element closest to the rock for models with the incident disturbance defined at the interface between rock and shotcrete. The initial velocity condition was set to zero (Nilsson 2009).

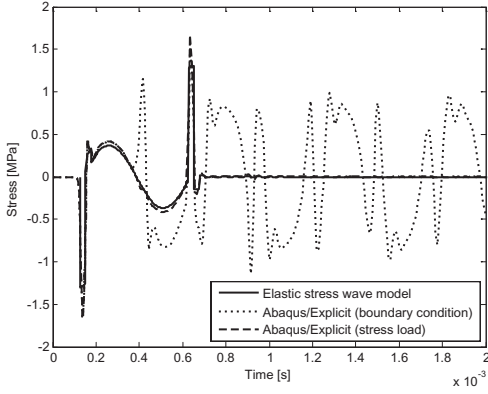


Figure 9. Stress response in the shotcrete element closest to the rock for models with the incident disturbance defined at the interface between intact and fractured rock for all models but the elastic stress wave model. The initial velocity condition was set to its maximum amplitude (Nilsson 2009).

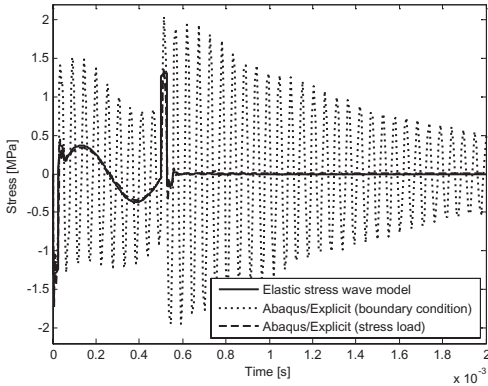


Figure 10. Stress response in the shotcrete element closest to the rock for models with the incident disturbance defined at the interface between rock and shotcrete. The initial velocity condition was set to its maximum amplitude (Nilsson 2009).

was made between the measured points. The time history of the measured accelerations furthermore contain quite some noise and only the first part of the signal up to the peak particle velocity was considered, see also Ansell (1999). For the velocity to return to zero the acceleration-time history was then mirrored, see Figure 2(b).

The elastic stress wave model and structural dynamic model were used to simulate the stress response within the shotcrete lining from the in situ acceleration-time history data from Figure 2(b). The stress responses in the shotcrete elements

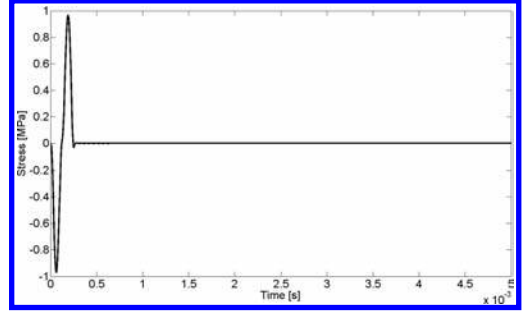


Figure 11. Stress response in the shotcrete element closest to the rock from in situ measured accelerations. Calculated with elastic stress wave model.

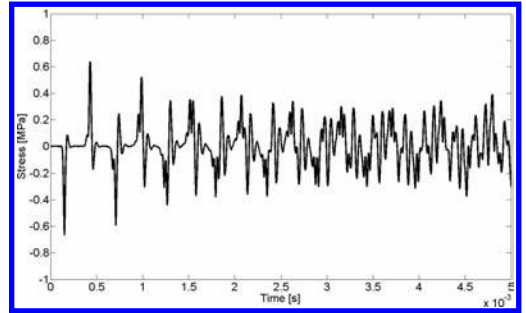


Figure 12. Stress response in the shotcrete element closest to the rock from in situ measured accelerations. Calculated with structural dynamic model.

closest to the rock surface are shown in Figures 11 and 12, respectively.

6 CONCLUSIONS

The three numerical models compared in this paper give similar results, with some exceptions. Despite the fact that the models only include wave propagation in one dimension the results are sometimes rather complex and difficult to interpret. The definition of the incoming stress wave must be adapted to each specific case studied and model used. The results do, however, show that it is possible to obtain similar results using the models for one-dimensional cases. It is important to note that the results from the older stress wave model and structural dynamic model also can be obtained with simple models created with the sophisticated FE program Abaqus/Explicit. It is therefore possible to continue the model development using FE models, while being able to compare the results to previous results.

6.1 Discussion

The comparison between the results from the tested models show good agreement in time and an acceptable coincidence in stresses when the incoming waves start from the zero level. This can be seen in Figures 7 and 8 from where it is also clear that the methods give different results depending on the definition of the load and place of insertion in the models. The elastic stress wave model is loaded by a stress wave that gives results that coincide with those from the similarly loaded Abaqus model. Application of the load as a boundary condition in Abaqus results in stresses that are very similar to those from the structural dynamic model.

It should be noted that there is a difference in time between when the stresses first appear in Figures 7 and 8, respectively, and this is due to the time it takes for the waves to propagate through the 0.5 m of rock. The lower stress-curves seen in Figure 8 are a result of the application of the loads at the interface between the rock and shotcrete. The structural dynamic model for such a case only consists of the shotcrete part, which gives a small mass and a relatively large stiffness. The model is thus stiffness dominated which can be seen in the high frequency vibrations that are superimposed over the stress curves, all the way until 0.5 ms. It is the positive stresses that are of interest here as they represent tensile stresses between rock and shotcrete, i.e. the adhesive bond stress. The comparison should therefore be done firstly with respect to the first arriving stress maxima.

When the stress load, calculated from velocities of the incoming wave, is suddenly applied and followed by a cosine-variation, the results become more complex but basically still comparable with those from the first load case. It is not practical to use such a load with the structural dynamic model and this was therefore omitted in the comparison made in Figures 9 and 10. These figures show large, sharp peaks with opposite signs, superimposed on sine-shaped stress waves similar to those in Figures 7 and 8. Such peaks correspond to a compressive impulse that is a result of the sudden increase in stress as the load is applied. When the load is applied as a boundary condition at the interface between rock and shotcrete this will dominate the results obtained from the Abaqus-model, as seen in Figure 10. It can be concluded that the maximum tensile stresses in principle coincide for the three models and for the different ways of applying the loads but that there exist superimposed disturbances and peaks that originate from dynamic effects.

The results obtained with measured accelerations demonstrate how in situ data can be used to evaluate the risk for bond failure between rock

and shotcrete by using the elastic stress wave and structural dynamic models, respectively. The results in Figure 11 show that the response calculated with the stress wave model in this case kept the harmonic form of the input wave in Figure 2(b). This is not the case for the results from the structural dynamic model in Figure 12 where it can clearly be seen that the dynamics of the mass-spring system affects the results, as was the case in Figure 8. The in situ acceleration data used has previously been evaluated with the stress wave model by Ansell (1999), including all results from the four test sites. Using a future three dimensional finite element method it will be possible to simultaneously include data that describe accelerations in two or three directions at many points of the model.

6.2 Ongoing research

The ongoing research aim is to extend the finite element models in Abaqus/Explicit to describe three-dimensional geometries and displacements. Linearly elastic material models of dynamically loaded shotcrete have been described, evaluated and compared and non-linear properties and behaviour of the rock and shotcrete will also be introduced in these models. The point detonation can possibly be modelled as a cavity within the rock with an impulsive pressure load on its surface. To allow the stress wave to disperse a rather large volume of rock will have to be modelled, depending on the distance to the detonation of explosives. For evaluation of the models further in situ acceleration measurements will be undertaken. The aim is to describe the wave propagation along tunnel walls following blasting at a tunnel front. The further work will also study newly sprayed and hardening shotcrete. The goal is detailed dynamic analyses using 3D models.

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REFERENCES

- Ansell, A. 1999. *Dynamically loaded rock reinforcement*, Doctoral thesis, Bulletin 52. Stockholm: KTH Structural Engineering.
- Ansell, A. 2004a. In situ testing of young shotcrete subjected to vibrations from blasting. *Tunnelling and Underground Space Technology*, 19(6): 587–596.

- Ansell, A. 2004b. A finite element model for dynamic analysis of shotcrete on rock subjected to blast induced vibrations. In Bernard (ed.), *Second International Conference on Engineering Developments in Shotcrete*, Cairns, 4–6 October 2004. Rotterdam: Balkema.
- Ansell, A. 2005. Recommendations for shotcrete on rock subjected to blasting vibrations, based on finite element dynamic analysis. *Magazine of Concrete Research*, 57(3): 123–133.
- Ansell, A. 2007. Shotcrete on rock exposed to large-scale blasting. *Magazine of Concrete Research*, 59(9): 663–671.
- Ansell, A. & Holmgren, J. 2001. Dynamically loaded young shotcrete linings. In Bernard (ed.), *International Conference on Engineering Developments in Shotcrete*, Hobart, 2–4 April 2001. Rotterdam: Balkema.
- Ansell, A. & Malmgren, L. 2003. The response of shotcrete to blast induced vibrations in mining. In Handley & Stacey (ed.), *Technology roadmap for rock mechanics, Proceedings of the 10th Congress of the ISRM*, Sandton City, 8–12 September 2003. Johannesburg: SAIMM.
- Beauchamp, K.J. 1995. *Impact testing of large scale shotcrete panels*, Master thesis. Waterloo: University of Waterloo.
- Bodare, A. 1997. *Jord- och Berdynamik (Soil and Rock Dynamics*, in Swedish). Stockholm: KTH Dept. of Soil and rock Mechanics.
- Dowding, C.H. 1996. *Construction Vibrations*. Upper Saddle River: Prentice-Hall.
- James, G. 1998. *Modelling of young shotcrete on rock subjected to shock wave*, Master thesis. Stockholm: Royal Institute of Technology.
- Kendorski, F.S., Jude, C.V. & Duncan, W.M. 1973. Effect of blasting on shotcrete drift linings. *Mining Engineering*, 25: 38–41.
- Mathworks. 2009. <http://www.mathworks.com/products/matlab/>
- McCreath, D.R., Tannant, D.D. & Langille, C.C. 1994. Survivability of shotcrete near blasts. In Nelson & Laubach (ed.), *Rock Mechanics*. Rotterdam: Balkema.
- Nakano N., Okada S., Furukawa K. & Nakagawa, K. 1993. Vibration and cracking of tunnel lining due to adjacent blasting (in Japanese, Abstract in English). *Proceedings of the Japan Society of Civil Engineers*, 462: 53–62.
- Nilsson, C. 2009. *Modelling of dynamically loaded shotcrete*, Master thesis 276. Stockholm: KTH Structural Engineering.
- Simulia. 2009a. http://www.simulia.com/products/abaqus_standard.html
- Simulia. 2009b. http://www.simulia.com/products/abaqus_explicit.html
- Tannant, D.D., Kaiser, P.K. & McCreath, D.R. 1996. Impact tests on shotcrete and implications for design for dynamic loading: field tests and modelling. In Aubertin, Hassani & Mitri (ed.), *Proceedings of 2nd North American Rock Mechanics symposium*, Montreal, 19–21 June 1996. London: Taylor and Francis.
- Tannant, D.D., Kaiser, P.K., McCreath, D.R., McDowell, G.M. & Beauchamp, K. 1995. Large-scale impact tests on shotcrete panels. In *Proceedings of the 12th CIM Mine Operators Conference*, Timmins.
- Tannant, D.D. & McDowell, G.M. 1993. *Dynamic testing of shotcreted drifts*. GRC Internal Report 93-12-IR. Sudbury: Laurentian University.
- Wood, D.F. & Tannant, D.D. 1994. Blast damage to steel fibre reinforced shotcrete. In Banthia & Mindess (ed.), *Fiber Reinforced Concrete—Modern Developments*. Toronto: UBC Press.
- Wersäll, C. 2008. *Blast-induced vibrations and stress field changes around circular tunnels*, Master thesis. Stockholm: KTH Dept. of Soil and rock Mechanics.

Structural behaviour of shotcrete on irregular hard rock surfaces

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ABSTRACT: Tunnels and underground openings in hard rock are often constructed with arch-shaped ceilings and complicated 3D geometries arise at intersections with cross tunnels and other openings. This is further complicated due to the often irregular shape of the rock surface and the uneven shotcrete thickness, making it difficult to analyse the interaction between rock and support systems. A study of the interaction in 3D between an irregular, rough rock surface and supporting rock bolts and shotcrete has begun, through finite element modelling using non-linear material models and formulations capable of describing large deformations. Various cases that may cause failure are to be studied, such as falling blocks or heavily jointed rock, bolt failure and drumminess between rock and shotcrete. An important conclusion from the preliminary results is that rock bolts should be placed at points where the shotcreted surface is locally convex, i.e. at peaks.

1 INTRODUCTION

In underground construction and tunnelling, the need for more efficient use of materials and sustainable designs require an increased understanding of the interaction between the rock and rock reinforcement systems of shotcrete. Tunnels and underground openings in hard rock are often constructed with arch-shaped ceilings that carry the weight of the above rock. Complicated three dimensional (3D) geometries arise at intersections with cross tunnels and other openings making it difficult to analyse the interaction between rock, shotcrete and other support elements such as rock bolts. This is further complicated due to the often irregular shape of the rock walls and the uneven shotcrete thickness. The design of shotcrete based rock support systems for such geometries must therefore be done with advanced analysis tools, such as based on the finite element (FE) method.

The division of Concrete Structures at KTH have been involved in research on shotcrete as rock reinforcement since the middle of the 1970s, see e.g. Holmgren (1979, 1985, 1992). One of the doctoral projects studied the load carrying capacity of bolt anchored shotcrete on rock and it was demonstrated by Nilsson (2004) and Nilsson & Holmgren (1999, 2001) that a further, detailed study of the interaction in 3D between an irregular, rough rock surface supported by bolts and shotcrete with varying thickness is of great interest. The numerical investigation must be made using FE models with non-linear material formulations capable of describing large deformations as well as concrete cracking and crushing. Of special interest is

the load carrying contribution from the bonding between rock and shotcrete. Various cases of local instability that may cause failure must be studied, such as falling blocks, heavily jointed rock, bolt failure and drumminess between rock and shotcrete. Time-dependent effects such as shotcrete creep and shrinkage, varying temperatures and humidity are also of interest. There is a need for geometric in situ data and experimentally obtained material data for structural shotcrete.

The aim of this recently initiated research project is to obtain an understanding of the performance of load carrying shotcrete systems for rock support. The main objective is to obtain an increased understanding of the behaviour of shotcrete as a construction material, with a focus on the entire life cycle from spraying until possible failure occurs. This paper summarizes the background to the project and the current knowledge, presents preliminary results and describes the ongoing work within the project.

Reliable and efficient analyses are needed since the construction of tunnels and other structures in rock are large investments for society and of great importance for infrastructure. As an example, there is today ongoing Swedish tunnelling and rock engineering projects for a cost of 1.5×10^9 EUR/year. The outcome of this project will lead to new guidelines and recommendations for practical use that will facilitate future construction of optimized, more efficient and economic tunnels and subspace structures. This will be of great importance in civil engineering underground work, tunnelling and also for the mining industry. An improved understanding of the interaction between the rock and rock

reinforcement systems of shotcrete will be of great value when designing sustainable infrastructure systems and will also lead to a more efficient use of the building materials and result in constructions that can maintain a high degree of safety without unnecessary repairs and rehabilitation.

2 SHOTCRETE AS ROCK REINFORCEMENT

Rock reinforcement is necessary to create stable underground structures in rock, e.g. tunnels and other subspace openings. More than one material or element is often included in the support systems, such as shotcrete, cast in place concrete, rock bolts, arches and other pre-fabricated structural elements. The structural principle is to reinforce the rock so that it can carry its own weight, even when an opening has been created within the rock mass. The interaction between rock, shotcrete and other reinforcing elements is complex and factors such as geometry, shape and irregularities of surfaces, built-in stresses, systems of cracks in the rock and material strength affects the load carrying capacity and deformability.

2.1 Geometries in 3D

Curved rock surfaces, such as tunnel roofs, will affect the load carrying capacity of rock-shotcrete systems. Sharp edges, peaks and depressions in the rock surface will provide anchorage between rock and shotcrete and lead to varying degrees of interaction and constraint as increasing load and stresses in the shotcrete approach the ultimate stress. The situation is further complicated by the often irregular shape of the rock walls and the varying shotcrete thickness which is a result of this. Shotcrete fills out holes and depressions in the rock resulting in a shotcrete surface that is smoother (harmonic-shaped) than the underlying rock, e.g. sine-shaped. This can lead to the formation of thin, critical shotcrete sections with lower load carrying capacity and an increased risk of crack formation. Rock bolts with washers (plates) give rise to stress concentrations in the shotcrete and it is therefore interesting to further study how the placement and properties of the bolts and washers affects the overall stress situation in the shotcrete.

Different cases of rock instability will define the design load cases for the rock support systems. A shotcrete lining sprayed onto an arch-shaped tunnel ceiling can be loaded by loose pyramid-shaped blocks of rock. Such blocks are the results of existing cracks in the rock mass and the pressure from the surrounding rock may push these against

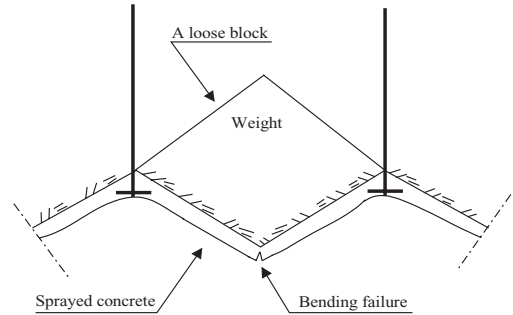


Figure 1. Failure in a bolt-supported shotcrete lining on rock (Chang 1994).

the shotcrete with possible punching failures as a result. Such a case is shown in Figure 1 where bending failure occurs in a bolt-supported shotcrete lining due to punching from a loose block. When a block or section of rock is severely fractured the tensile strength and bending stiffness are reduced to zero. This mass will act as a flexible surface load on a supporting shotcrete lining as it is deformable and therefore follows the displacements that occur in the shotcrete. The size of the load is given by the inner friction angle and the density of the material.

2.2 Irregular rock surfaces

The example shown in Figure 1 also demonstrates how the irregular shape of a rock surface can lead to bending and stress concentrations in shotcrete linings. The properties of irregular shotcrete surfaces have been investigated in laboratory tests by Chang (1994) who also used numerical modelling to evaluate the results. The tested shotcrete models were given sine-shaped surfaces. The importance of the shape of the shotcrete surface has also been a part of the work by Malmgren (2005). In numerical analyses with 2D models, that also included rock bolts, the shotcrete was given a saw-toothed variation and the importance of changes in shotcrete thickness was also studied to some extent. An example of a tunnel profile with irregular shape, based on in situ measurements, is shown in Figure 2. Another study on the topic has been presented by Fotieva & Bulychev (1996) who approached the problem with small scale analytical formulations. They concluded that variation in shotcrete thickness leads to stress concentrations and an increased risk of crack initiation. This has also been observed in situ by Ansell (2004) and Ansell & Slunga (2004) during failure mapping of shotcrete within the traffic tunnels of the Southern Link (Södra länken) in Stockholm. Shrinkage

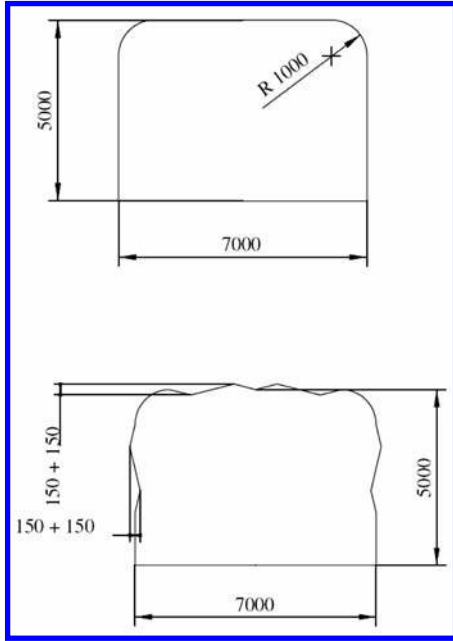


Figure 2. Tunnel with smooth rock surface and with a roughness of 150 mm (Malmgren 2005).

cracks often appeared at sections with thin shotcrete close to much thicker parts.

3 FINITE ELEMENT MODELS

There is little published research on numerical studies of rock supported with shotcrete and bolts where non-linear material formulations for concrete and steel are used (Ansell 2009). It is today usually not possible to describe deformation of support systems up to crack initiation and failure. The focus is often set on the properties of the rock and the reinforcement is often given only elastic deformation properties, for example as in the recent studies by Karakus (2007) and Chen et al. (2009). There are some exceptions, e.g. from Golser (1999) and Liu et al. (2008), and in those cases a powerful, general FE program with built-in advanced material formulations such as Abaqus/Standard (Simulia 2009) have been used.

3.1 Irregular 2D slabs

Models in 3D are needed for detailed studies of the interaction between rock, shotcrete and other support. One simple model is a horizontal slab of shotcrete that carries a severely cracked rock mass, as described in section 2.1. Such a model will only

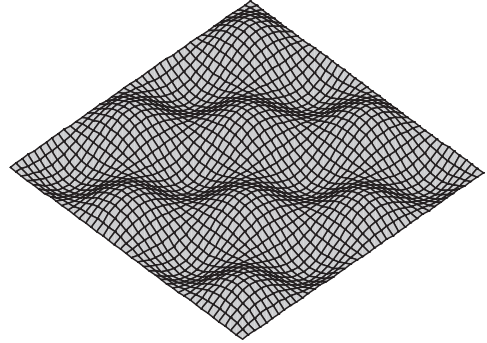


Figure 3. Finite element model of a plane shotcrete slab with harmonic irregularities.

include the shotcrete, and possibly also rock bolts, but the rock will be present as a uniformly distributed load applied on top of the slab. The easiest way to approximate irregularities such as those shown in Figure 2 is to apply harmonic (sine-shaped) variations in two orthogonal directions to the surface of this otherwise plane and horizontal slab. The result is shown in Figure 3 and can be seen as a model of a part of a shotcreted horizontal tunnel ceiling. These types of shell structure were modelled by Nilsson (2004) who studied slabs with freely supported and fixed edges. The effect of one or many rock bolts was also included by restricting the vertical movements for a small number of nodes before the load was applied. Sharp irregularities can in this way thus be approximated with harmonically varying surfaces. It should be noted that this shape is a better approximation of the outer surface of the shotcrete than of the rock surface. However, different upper and lower surfaces require varying shotcrete thickness which is not included in this simplified approximation.

3.2 Shotcrete models in 3D

A further development of the modelled plane slab is a curved shotcrete shell with a semi-circular cross-section. In its most simple form it will have a constant thickness and harmonic irregularities, such as shown in Figure 4. These types of shells can also be suspended using rock bolts but will also carry load through the arch effect. A more sophisticated version is shown in Figure 5. The outer surface of the curved shell is here given a highly irregular, randomly distributed shape which is more similar to profiles such as in Figure 2. The inner surface of the shell is completely smooth, thus approximating the outer surface of shotcrete that fills out depressions in the rock. Such a shotcrete shell will

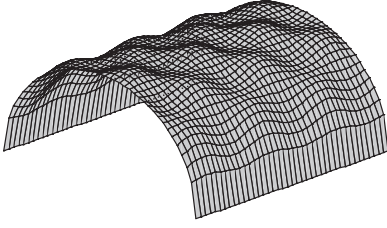


Figure 4. Finite element model of a shotcrete shell with harmonic irregularities.

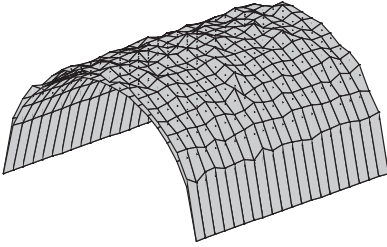


Figure 5. Finite element model of a shotcrete shell with a highly irregular outside and a smooth inside.

thus have a varying thickness and is a reasonable approximation of what can be observed in situ.

3.3 Models with rock and shotcrete

It is possible to describe the interaction between punching loads from loose blocks with the shell models described above. The bonding between rock and shotcrete can also be included by fixing all the nodes in the direction orthogonal to the shell surface. The rock will, however, give no contribution to the load carrying capacity of the system. For detailed studies and more complicated load cases the rock must therefore be directly included in the models. Models in 3D including rock and rock support are often done in a large scale with a focus on the deformation of the rock mass directly around a tunnel or other opening. One such example is presented by Brandshaug & Rosengren (2008) who studied the effect of blasting in rock tunnels. The rock support included consists of bolts and shotcrete described with only linear material models, also with a smooth rock surface and a constant shotcrete thickness. A detailed study of the non-linear deformation of bolts and shotcrete would be too computationally expensive with such a large model. Results from large-scale modelling of tunnels in large volumes of rock can, however, be used as boundary conditions for smaller models, in local scale. Such 3D models can be used for detailed studies of the

shotcrete-rock interface, also including rock bolts and other reinforcement.

4 MATERIAL PROPERTIES

Modern concrete technology makes it possible to adapt the composition of shotcrete to the performance needed. Changes in material properties will have a great impact on the load carrying capacity of shotcrete-based reinforcement systems. The effects of steel and synthetic fibre reinforcement can also be included in the models and if needed also bar- and mesh-reinforcement. For concrete (shotcrete) and steel the stress-strain relations must be described with non-linear material formulations that account for plastic deformation in steel and cracking in concrete materials. Research on non-linear material models for shotcrete has been published by e.g. Meschke et al. (1996). That particular model describes the strain in the shotcrete as a sum of elastic strain, shrinkage, temperature dependent strain, visco-plastic strain and strain due to aging.

4.1 Non-linear behaviour of shotcrete

The analyses within this project have been carried out using the general finite element program Abaqus/Standard (Simulia 2009) which contains linear and non-linear material formulations suitable for concrete, also in combination with other materials. The non-linear model chosen for the analysis is based on the “smeared crack approach” which means that no individual cracks are initiated in a structure but there will locally be areas with decreased stiffness.

The material properties for one type of fibre reinforced shotcrete used in the analyses are given in Table 1. The uniaxial stress-strain relationship in compression and tension is governed by the piece-wise linear curve shown in Figure 6. On the compressive side (negative σ) the first line represents elastic deformations up to the elastic limit f_{cy} . The inelastic part is described by three lines before the ultimate compressive strength f_c followed by another five lines until the maximum compressive strain is reached. The tensile stresses (positive σ) follow the linear, elastic relation until crack failure at f_t . The response after cracking in Abaqus/Standard (Simulia 2009) is governed by a relation based on fracture mechanics. A stress-displacement relation approximated with a linear function is used, for the analyzed shotcrete shown in Figure 7. The area under the curve represents the fracture energy $G_F = 0.5u_0f_t$ where u_0 is the crack opening displacement. Typical values for G_F representing concrete is within 50–200 N/m and in this case was a value

Table 1. Material properties for the analyzed fibre reinforced shotcrete.

Modulus of elasticity	E	34 GPa
Poisson's ratio	ν	0.2
Compressive strength	f_c	50 MPa
Elastic limit	f_{cy}	30 MPa
Tensile strength	f_t	4.5 MPa
Fracture energy	G_F	135 N/m
Crack opening displacement	u_0	0.06 mm

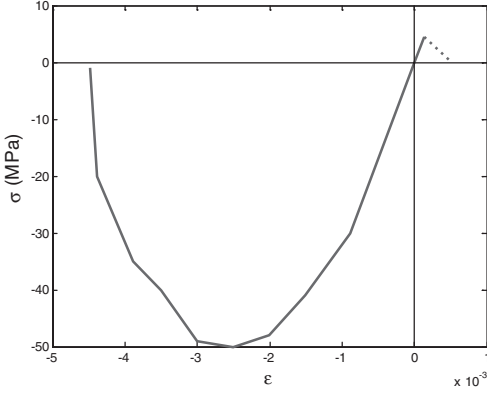


Figure 6. Uniaxial stress-strain relationship for the analyzed fibre reinforced shotcrete.

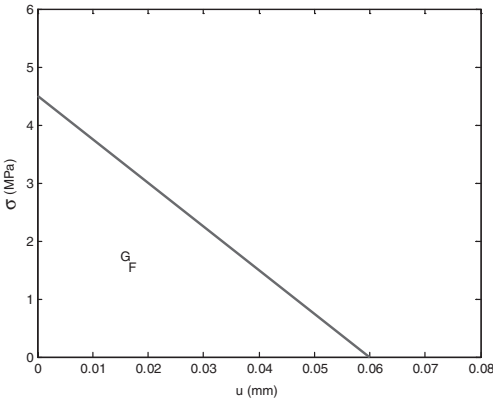


Figure 7. Tensile stress-displacement relationship for the analyzed fibre reinforced shotcrete.

representative for (sprayable) concrete containing steel fibres chosen, see Nilsson (2004).

4.2 Steel

The steel material in rock bolts, bars and mesh reinforcement can be described using linear or non-linear stress-strain models, depending on the

expected stress levels in the steel. For rock bolts, the non-linear model is justified for cases with weak bolts and concentrated, large deformations around single bolts. This usually requires a ductile shotcrete, reinforced with e.g. large amounts of steel fibres or steel mesh. The use of stiff steel washers, quadrilateral or circular steel discs, is necessary for the transformation of load from rock to bolts, via the shotcrete lining. In a case with strong rock bolts failure will occur in the shotcrete, without plastic deformations in the bolts which thus can be defined as only linearly elastic. This is demonstrated in the preliminary results presented in the following sections.

The reinforcing effect from evenly distributed steel fibres can be included by increasing the ductility of the shotcrete, i.e. changing G_F and u_0 , as pointed out in the previous section. Steel mesh and discrete reinforcing steel bars can be included as embedded elements in the shotcrete and it is also possible in Abaqus/Standard (Simulia 2009) to define the degree of coupling between the two materials. The stress-strain relationships can be either linear or non-linear, the choice should be based on the levels of strain expected. An elastically linear model for the reinforcement may give an over-stiff behaviour for cases with large deformations.

4.3 Rock

In the presented preliminary examples the rock only acts as a load and does not contribute to the supporting system. This case is relevant for a severely fractured rock with a zero modulus of elasticity, giving an evenly distributed surface load on the supporting shotcrete lining. In later more detailed modelling examples, the properties of the rock mass will be described based on realistic material data including system of cracks, the possible geometry of loose blocks, properties of cracks and the arch shape of the analyzed tunnel or opening. A jointed rock mass will be described as built up by blocks that are loose or connected to "solid rock". The deformation of each block will be elastic while the rock joints will be described using elasto-plastic laws based on rock mechanic theories. In the studied models at a "local" scale the rock will be solid and elastic but with one or more loose blocks held in place by shotcrete, rock bolts and possibly also other reinforcing elements. Displacement of blocks can be described based on experience or results from large-scale rock mechanic models.

5 PRELIMINARY RESULTS

As a first part of the project preliminary results from the work by Nilsson (2004) are included.

An important part of that investigation was a study of the importance of placement of rock bolts with respect to the load carrying capacity and risk of early shotcrete failure. The examples studied include a horizontal, quadrilateral section of shotcrete suspended with single or multiple rock bolts. The thickness of the shotcrete was set to 40 or 80 mm and the irregular surfaces were as described in Figure 3. The shotcrete lining was modelled with 3D shell elements with four nodes and nine integration points through the thickness, referred to as S4R5 in Abaqus/Standard (Simulia 2009). The uniformly distributed load from the weight of the rock was applied as vertical point-loads at every node of the model. The load-deformation behaviour and the ultimate load were calculated by step-wise increasing the load.

5.1 Irregular slab with one rock bolt

The first studied section of shotcrete is a quadrilateral $4 \times 4 \text{ m}^2$ slab with a bolt placed at the centre. The finite element model with 40×40 shell elements is shown in Figure 8. The irregular surface is sine-shaped with a “wave length” of 1.6 m. No displacements or rotations are allowed at the edges of the slab, thereby accounting for the influence from adjacent sections of a large, continuous shotcrete slab. The rock bolt with a quadrilateral washer is represented by 9 nodes that are fixed in the vertical direction. Two cases were studied, with the bolt placed either at a peak or at a depression in the slab, as shown in Figure 9.

The relative stiffness of the shotcrete slab is here defined as the ratio between the ultimate deflection of a similar but plane slab δ_0 and the analyzed irregular slab δ_i . For the presented case the results are given in Figure 10, for 40 and 80 mm shotcrete and heights of the irregularities up to 400 mm. The

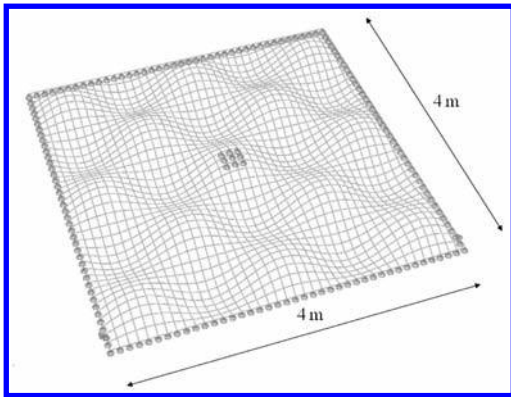


Figure 8. Finite element model of shotcrete and one rock bolt (Nilsson 2004).

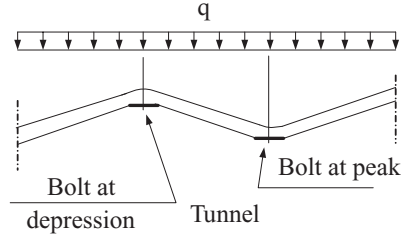


Figure 9. Placement of rock bolts at peaks or in depressions (Nilsson 2004).

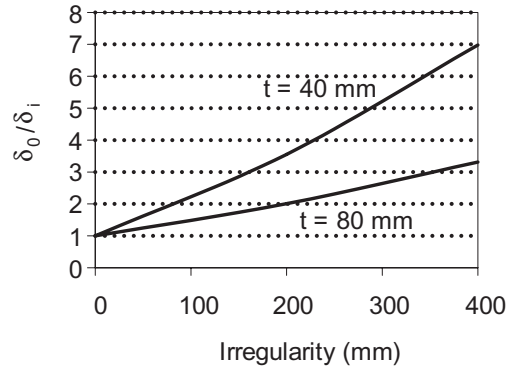


Figure 10. Relative stiffness as a function of irregularity and shotcrete thickness (Nilsson 2004).

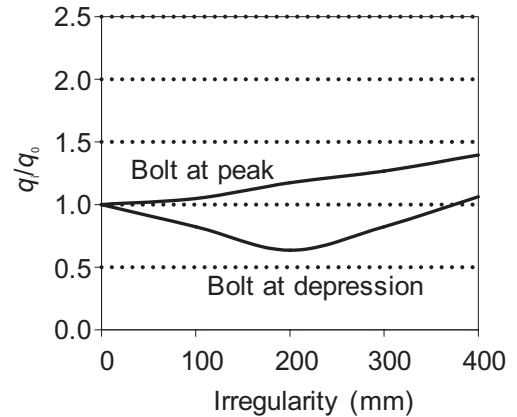


Figure 11. Failure load for a 80 mm thick shotcrete slab as function of irregularity and placement of rock bolt (Nilsson 2004).

relative load carrying capacity can in the same way be defined as the ratio between the corresponding maximum distributed loads q_0 and q_i , respectively, at which failure occurred. The results are shown in Figure 11, for the cases of the bolts being placed at a peak and in a depression of the slab. The stress

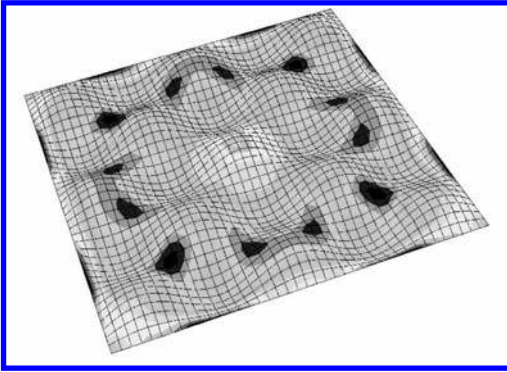


Figure 12. Principal stress distribution on the upper surface of a 80 mm thick shotcrete slab. High tensile stresses around the bolt placed in a depression within the slab (Nilsson 2004).

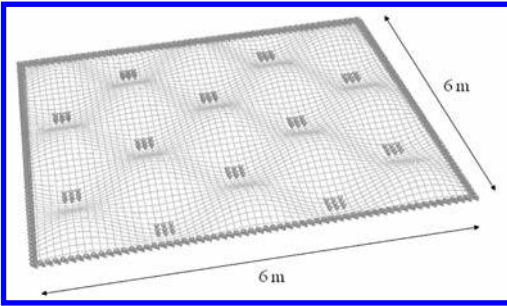


Figure 13. Finite element model of shotcrete and twelve rock bolts (Nilsson 2004).

distribution in a slab is represented by Figure 12 which shows the principal stresses over the upper surface of a 80 mm thick shotcrete slab with fixed edges and the rock bolt placed at a depression in the slab. The highest tensile stresses occur at the edges and around the bolt where tensile failure occurs.

5.2 Irregular slab with systematic bolting

This second example demonstrates the behaviour of a slab with multiple bolts placed systematically over the surface. The model of this slab shown in Figure 13 is $6 \times 6 \text{ m}^2$, with 60×60 shell elements and with twelve sets of fixed nodes, but otherwise identical to the slab in Figure 8. The height of the irregularities was set to 400 mm and the “wave length” was here 2.4 m. The load-deflection relations up to failure are shown in Figure 14, for 40 and 80 mm thick shotcrete slabs, respectively. The results also demonstrate the differences due to bolt placement, i.e. the entire set of bolts placed at peaks or in depressions. The stress distributions in the loaded slabs are here shown in Figures 15–16.

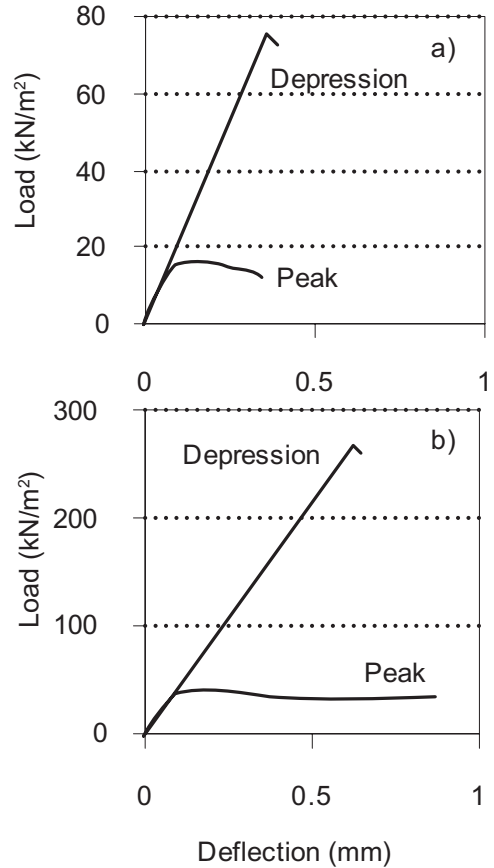


Figure 14. Load-deflection for shotcrete slab with multiple bolts and a thickness of a) 40 mm and b) 80 mm (Nilsson 2004).

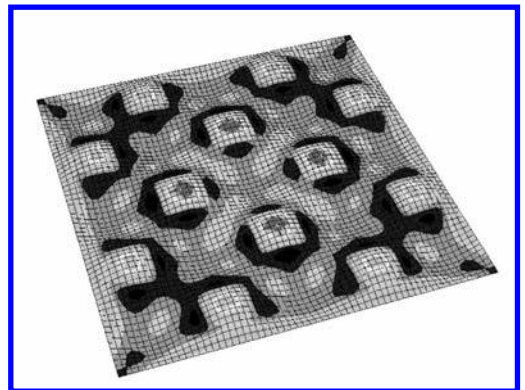


Figure 15. Principal stress distribution on the upper surface of a shotcrete slab suspended using twelve rock bolts. High tensile stresses around the bolts placed in depressions of the slab (Nilsson 2004).

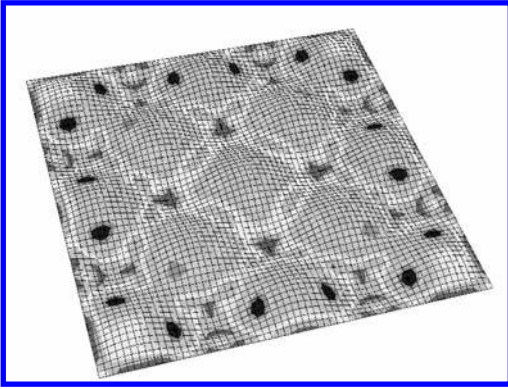


Figure 16. Principal stress distribution on the upper surface of a shotcrete slab suspended using twelve rock bolts. Low tensile stresses around the bolts placed at peaks on the slab (Nilsson 2004).

The highest tensile stresses occur for the case with the bolts placed within depressions (Fig. 15) while placement at a peak results in low tensile stresses around the bolts (Fig. 16). In the latter case there is high, concentrated compressive stresses on the lower side of the slab, at each bolt, i.e. a stress distribution as in Figure 15 but with opposite stress sign (+/-).

6 SUMMARY

The presented project has recently started and currently the preliminary results from a pilot-study are available. These results do, however, give a representative description of some of the features that are studied in detail within the ongoing project. The finite element models of rock and shotcrete surfaces will be based on realistic data, from measurements in situ and a side-project with in situ measurements is therefore being prepared. The data must capture the structure of the rock surface with holes, depressions and peaks clearly defined. It is also important to have accurate measurements of the outer shotcrete surface since the variation in shotcrete thickness is the difference between these two sets of geometric data. Geometric data can possibly be obtained with laser-scanning, but a review of previous measurements from large civil engineering projects is also done.

6.1 Preliminary results

The results from modelling of the shotcrete slab with one rock bolt presented in section 5.1 shows that the irregularities have a greater effect on the slab stiffness than on the load bearing capacity.

The structural response of the slab is affected by the thickness in relation to the irregularities so that a thin slab becomes relatively much stiffer with increased irregularity compared to a thicker slab. The reason for this is the relatively large change in bending stiffness as a function of irregularity for the thinner slab (Nilsson 2004). The placement of the bolt in a depression within the shotcrete resulted in tension across the entire shotcrete thickness around the bolt. This reduced the load bearing capacity compared to a case with the bolt placed at a peak. It was also observed by Nilsson (2004) that the maximum load capacity of an irregular slab was reached at a smaller deflection compared with a plane slab of identical dimensions, especially when the bolt was placed at a depression.

The examples in section 5.2 show that the load bearing capacity increases if bolts are located at peaks instead of depressions. It should be noted that these results demonstrate this for sets of evenly distributed bolts, with all bolts placed either at peaks or at depressions in the shotcrete. The distribution of the principal stresses given in Figures 15–16 show that large concentrated tensile stresses develop when the bolts are located at the depressions and lower stresses develop when placed at peaks. A change in bolt placement resulted in completely different structural behaviour, as demonstrated in Figure 14. The 40 mm thick slab with bolts at the peaks show an almost linear response up to the maximum load whereas the slab with bolts at depressions begins to soften at a considerably lower load. For the 80 mm slab the behaviour is similar but the load level is much higher. It should be noted that the load bearing capacity of the 40 mm slab with bolts at the peaks is twice that of the 80 mm slab with bolts at depressions.

It is commented by Nilsson (2004) that when the bolts are placed at the peaks the load is basically carried by compressed domes between the bolts, thereby increasing the load bearing capacity. It is also pointed out that these high values increase the risk of tensile failures in the bolts and punching failures of the bearing plates. The most economical way to avoid this problem must be to increase the dimensions of bolts and washers instead of the thickness of the shotcrete.

6.2 Ongoing research

The ongoing work focuses on the behaviour of irregular shotcrete slabs and shells and the effect of varying shotcrete thickness (Beijer-Lundberg 2009). The following parts of the project will deal with the interaction between rock and reinforcement consisting of shotcrete and rock bolts, studied through detailed modelling in 3D including variations in rock geometries, shotcrete thickness,

material descriptions, etc. The models of the rock-shotcrete system describe sections with 3D geometry, based on the principles outlined here in section 3, but with increased complexity. The models will include local instabilities in the rock, such as loose rock blocks, seriously cracked rock masses, rock bolt failure, and partial debonding between shotcrete and rock. The behaviour of rock-shotcrete systems are simulated all the way until failure occurs and the tested constitutive material models must therefore be able to account for large deformations in 3D. Reinforcing steel and other reinforcement will be included in some models which will facilitate a detailed representation of stresses, deformations, damage and cracks within the shotcrete and at the interface between shotcrete and rock. The applied loads, the geometries of tunnels and underground openings, etc will be chosen on basis of conditions that are common in tunnelling and civil engineering.

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REFERENCES

- Ansell, A. 2004. *Utredning angående sprickor i sprutbetong på dräner i Södra Länken—Analys av trolig skadeorsak (Investigation of cracked shotcrete on drains in the Southern Link—Analysis, in Swedish)*, PM 2004-01-19. Stockholm: Vägverket.
- Ansell, A. 2009. *3D-Modelling of the interaction between rock and shotcrete* (in Swedish, abstract in English), Report K33. Stockholm: Stiftelsen Svensk Bergteknisk Forskning.
- Ansell, A. & Slunga, A. 2004. *Utredning angående sprickor i sprutbetong på dräner i Södra Länken—Sprickbild i fält (Investigation of cracked shotcrete on drains in the Southern Link—Mapped cracks, in Swedish)*, PM 2004-01-20. Stockholm: Vägverket.
- Beijer-Lundberg, A. 2009. *A parametric study of irregularly shaped shotcrete shells*, Master thesise (in prep.). Stockholm: KTH Structural Engineering.
- Brandshaug, T. & Rosengren, L. 2008. *3D Numerisk analys av explosionslaster i bergtunnlar (3D Numerical analysis of explosions in rock tunnels, in Swedish)*, SveBeFo Rapport 89. Stockholm: Stiftelsen Svensk Bergteknisk Forskning.
- Chang, Y. 1994. *Tunnel support with shotcrete in weak rock—A rock mechanic study*, Doctoral thesis, Report 94/1. Stockholm: KTH Soil and Rock Mechanics.
- Chen, S.-H., Fu, C.-H. & Isam, S. 2009. Finite element analysis of jointed rock masses reinforced by fully-grouted bolts and shotcrete lining. *International Journal of Rock Mechanics and Mining Sciences*, 46(1): 19–30.
- Fotieva, N.N. & Bulychiev, N.S. 1996. Design of sprayed concrete tunnel lining upon static and seismic effects. In Barton, Kompen & Berg (ed.), *Proceedings of the second International Symposium on Sprayed Concrete*, Gol, 23–26 September 1996.
- Golser, J. 1999. Behaviour of early-age shotcrete. In: Celestino & Parker (ed.), *Shotcrete for Underground Support VIII*, São Palo, 11–15 April 1999. Reston: ASCE Publications.
- Holmgren, J. 1979. *Punch loaded shotcrete linings on hard rock*, Doctoral thesis, Report 7:2/79. Stockholm: Stiftelsen Bergteknisk Forskning.
- Holmgren, J. 1985. *Bolt anchored steel fibre reinforcement linings*, Report 73:1/85. Stockholm: Stiftelsen Bergteknisk Forskning.
- Holmgren, J. 1992. *Handbok i bergförstärkning med sprutbetong (Handbook in rock reinforcement with shotcrete, in Swedish)*. Vällingby.
- Holmgren, J. & Nilsson, U. 2001. Load bearing capacity of steel fibre reinforced shotcrete linings, In Bernard (ed.), *International Conference on Engineering Developments in Shotcrete*, Hobart, 2–4 April 2001. Rotterdam: Balkema.
- Karakus, M. 2007. Appraising the methods accounting for 3D tunnelling effects in 2D plane strain FE analysis. *Tunnelling and Underground Space Technology*, 22(1): 47–56.
- Liu, H.Y., Small, J.C. & Carter, J.P. 2008. Full 3D modelling for effects of tunnelling on existing support systems in the Sydney region. *Tunnelling and Underground Space Technology*, 23(4): 399–420.
- Malmgren, L. 2005. *Interaction between shotcrete and rock: Experimental and numerical study*, Doctoral thesis, Report 2005:48. Luleå: Luleå technical university.
- Meschke, G., Kropik, C. & Mang, H.A. 1996. Numerical analyses of tunnel linings by means of a viscoplastic material model for shotcrete. *International Journal for Numerical Methods in Engineering*, 39(18): 3145–3162.
- Nilsson, U. 2004. *Structural behaviour of fibre reinforced sprayed concrete anchored in rock*, Doctoral thesis, Bulletin 71. Stockholm: KTH Structural Engineering.
- Nilsson, U. & Holmgren, J. 1999. Design of steel fibre reinforced shotcrete linings In *Nordic Concrete Research Meeting*, Reykjavik. Oslo: Norsk Betongforening.
- Simulia. 2009. http://www.simulia.com/products/abaqus_standard.html

Crack widths in ASTM C-1550 panels

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ABSTRACT: Maximum permissible crack widths are an important issue for many Fibre Reinforced Shotcrete (FRS) tunnel linings due to the influence cracks have on post-crack strength, durability, and water ingress. The relationship between crack width and the deflection expected to occur in ground stabilized using a FRS lining is also of interest to geotechnical engineers. When performance assessment tests are undertaken on FRS samples it is therefore necessary to evaluate performance at relevant crack widths. Given the widespread use of ASTM C1550 panels for post-crack performance assessment of FRS the relationship between the central deflection measured in this test and the corresponding maximum widths of the three radial cracks that normally form is of significance. A closed-form solution is presented for the angles of rotation and corresponding maximum crack widths in the three radial cracks that form in a C1550 test based on their orientation relative to the mid-point bisectors between the supports. The effect of the characteristic probability distribution in crack locations on variations in crack width and the magnitude of the average crack width is also considered.

1 INTRODUCTION

Many aspects of the in-service performance of Fibre Reinforced Shotcrete (FRS) are related to crack widths. The most notable of these include the relation between crack width and residual strength (Banthia & Trottier, 1994), deflection, durability of the concrete matrix (Neville, 1995), and susceptibility of fibres within the concrete to corrosion (Kosa & Naaman, 1990; Nordström, 2001). In post-crack performance assessment tests for FRS based on beams and panels the maximum crack width experienced by the specimen is controlled by the geometry of the specimen and the angle of rotation exhibited at the hinges that form after cracking of the concrete matrix. In the ASTM C1550 round panel test (ASTM, 2008) the magnitude θ_i of the Crack Rotation Angles (CRAs) that arise at the three radial cracks are influenced by the locations of the cracks relative to the mid-point bisectors between the adjacent pairs of pivoted supports and the deflection imposed at the centre (Figure 1).

General characteristics of the failure modes in C1550 panels are described geometrically in Figure 2 for the case of three symmetrically occurring radial cracks and in Figure 3 for the general case of three radial cracks. Bernard & Xu (2008) recently presented an exact rigid plate analysis to determine the relation between crack location and the three CRAs. An approximate closed-form solution to this same problem is presented below together with

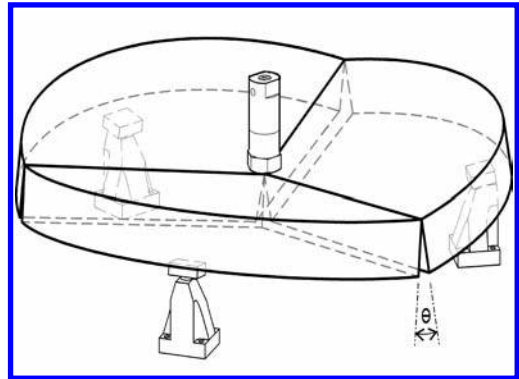


Figure 1. ASTM C1550 round panel test.

comparisons to the exact solution and a discussion concerning the magnitude of errors associated with the rigid plate approximation to structural behaviour in the uncracked portions of a C1550 panel.

In the C1550 test the position of each supporting pivot remains fixed for all specimens but the locations of the three radial cracks that form are uncontrolled. The position of each crack is characterized by a Mid-point Offset Angle (MOA) relative to the corresponding mid-point bisector, namely, angles α , β , and γ (Figure 3). Tran et al. (2005) used measurements of crack locations in about five hundred C1550 panels to find that the distribution of Midpoint Offset Angles can be described very well by the Weibull probability density function (PDF), viz.

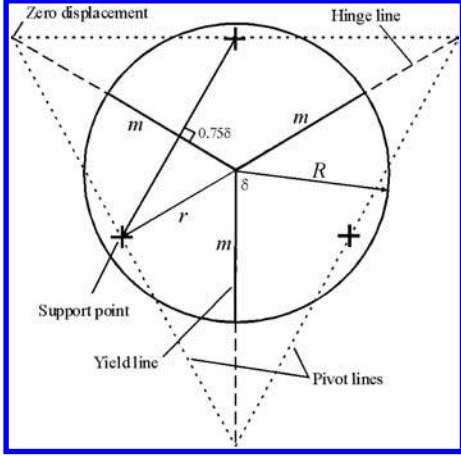


Figure 2. Folding plate analysis of ASTM C1550 panel with three symmetric radial cracks.

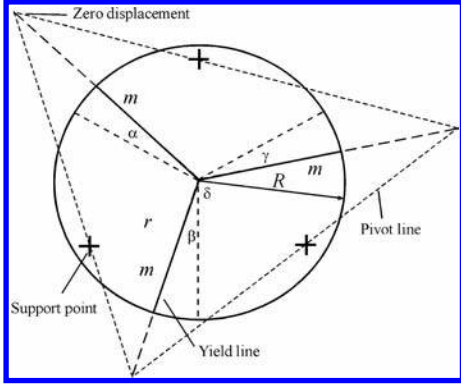


Figure 3. Folding plate Analysis of ASTM C1550 panel with three general radial cracks.

$$f(\alpha, \beta, \gamma) = \alpha_1 \beta_1^{-\alpha_1} \gamma^{\alpha_1 - 1} e^{-(\gamma/\beta_1)^{\alpha_1}} \quad (1)$$

where $\alpha_1 = 1.108$ and $\beta_1 = 13.038^\circ$. This PDF has presently been employed to conduct a Monte Carlo Simulation (MCS) to determine the characteristic distribution of the CRAs θ_α , θ_β and θ_γ for a sample population of C1550 panels. The CRAs were determined using both the exact analysis described by Bernard & Xu (2008) and the approximate closed-form method described in the Appendix. Random number generators were used together with the Weibull PDF to choose the sign and magnitude of α , β and γ , each of which were assumed to be independent. The summation $(\theta_\alpha + \theta_\beta + \theta_\gamma)$ produced as output from the MCS using the approximate closed-form solution was then assessed using Maximum Likelihood Estimation (Law & Kelton, 2000) and the Chi-square test (Kendall & Stuart,

1979) to determine which of three probability distributions (the Gaussian, Weibull, and Lognormal functions; Johnson et al., 1994) best described the outcome. The CRAs θ_α , θ_β and θ_γ were then used to estimate crack widths in the panels.

2 RESULTS

Using the Monte Carlo Simulation technique referred to above, the variation in $(\theta_\alpha + \theta_\beta + \theta_\gamma)$ with $(\alpha + \beta + \gamma)$ for a unit central deflection δ was found to be described by the distribution shown in Figure 4 (all angles in degrees). The quantity $(\theta_\alpha + \theta_\beta + \theta_\gamma)$ displayed a minimum equal to $3\sqrt{3}\delta_0/r = 0.794$ degrees for the symmetric pattern of radial cracks. This value is derived from a geometric analysis of the angle at each radial hinge that arises when a deflection δ is imposed at the centre (Tran et al., 2001).

As $(\alpha + \beta + \gamma)$ departed from zero, the quantity $(\theta_\alpha + \theta_\beta + \theta_\gamma)$ increased in magnitude. As $|\alpha| + |\beta| + |\gamma|$ departed from zero, $(\theta_\alpha + \theta_\beta + \theta_\gamma)$ also increased in magnitude (see Figure 5, all angles in degrees). However, $(\theta_\alpha + \theta_\beta + \theta_\gamma)$ appeared to be relatively insensitive to the magnitude of each MOA. When $|\alpha| + |\beta| + |\gamma| = 60^\circ$ we find that $(\theta_\alpha + \theta_\beta + \theta_\gamma)$ is, on average, 11% greater than the minimum obtained for the symmetric case, and for $|\alpha| + |\beta| + |\gamma| = 100^\circ$ we find that $(\theta_\alpha + \theta_\beta + \theta_\gamma)$ is, on average, 37% greater. This indicates that the approximate solution is less sensitive to the midpoint angles than the exact solution which indicated a 55% difference at $|\alpha| + |\beta| + |\gamma| = 100^\circ$ (Bernard & Xu, 2008). It must be noted, however, that based on the distribution represented by Equation (1), the likelihood that $|\alpha| + |\beta| + |\gamma| \geq 100^\circ$ is less than 0.01 percent.

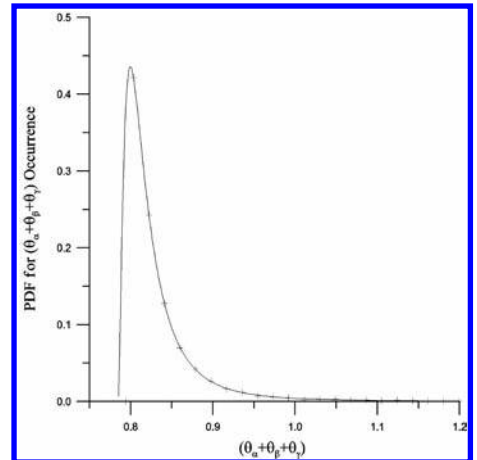


Figure 4. Occurrence frequency for $(\theta_\alpha + \theta_\beta + \theta_\gamma)$.

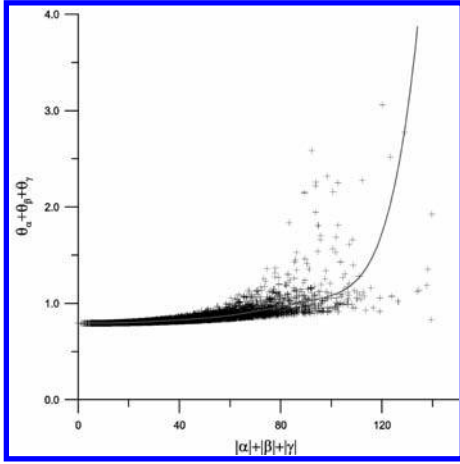


Figure 5. Magnitude of $(\theta_\alpha + \theta_\beta + \theta_\gamma)$ as a function of the sum of the midpoint angles.

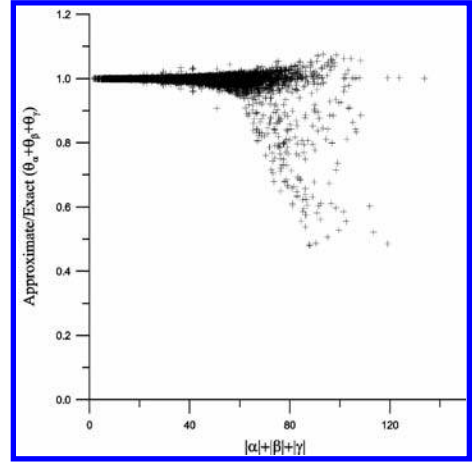


Figure 6. Ratio of approximate $(\theta_\alpha + \theta_\beta + \theta_\gamma)$ over exact values as a function of the sum of midpoint angles.

When the characteristic distribution of $(\theta_\alpha + \theta_\beta + \theta_\gamma)$ obtained by the MCS was assessed using the method of Maximum Likelihood Estimation to determine the best fitting Probability Distribution Function (PDF) the distribution was assessed for a simulated sample of specimens represented by Equation (1) (Figure 4). It was found that the three-parameter Lognormal distribution was the most suitable for describing the quantity $(\theta_\alpha + \theta_\beta + \theta_\gamma)$ since using this distribution achieved the maximum likelihood. The lognormal distribution gave a mean of 0.837 degrees, a standard deviation (SD) of 0.063 degrees, and a COV of 7.54% for $(\theta_\alpha + \theta_\beta + \theta_\gamma)$. The mean was therefore only 5.4% greater than the minimum associated with the symmetric cracking case. Moreover, the average for the approximate analysis was only 0.1% greater than for the exact analysis. This suggests that characteristic variations in crack location have, on average, a small influence on the sum of the CRAs experienced at the cracks compared to the symmetric case. Examination of Figure 5 and comparison with the results by Bernard & Xu (2008) for the exact case suggests that the magnitude of the variation in angles between the exact and approximate methods differs by a negligible amount for $|\alpha| + |\beta| + |\gamma| \leq 60^\circ$ (Figure 6). The difference becomes more significant when the cracks occur closer to the pivoted supports.

The Monte Carlo Simulation was conducted using estimates of Crack Rotation Angles obtained by the approximate method (see the Appendix) for a progressively increasing central deflection. The results indicated that the magnitude of the mean CRAs obtained using the approximate closed-form solution were greater than for the exact analysis presented by Bernard & Xu (2008), especially as

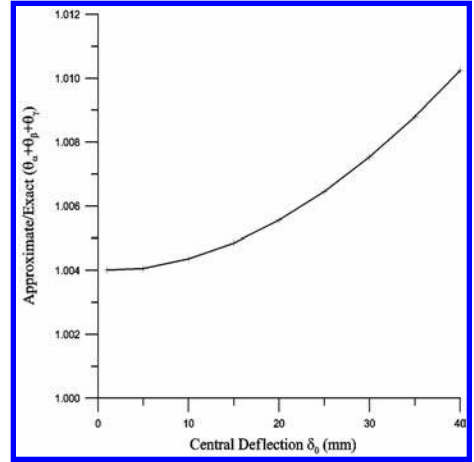


Figure 7. Ratio of approximate $(\theta_\alpha + \theta_\beta + \theta_\gamma)$ over exact values as a function of central deflection for a characteristic distribution of α , β , and γ . Each point represents the mean of a distribution of ratios at each given deflection.

the central deflection approached 40 mm, but the maximum mean error was nevertheless no greater than 1% (Figure 7). This error was due in part to the simplification $\theta_i \equiv \sin \theta_i$ that was incorporated into the approximate method since $\theta_i \geq \sin \theta_i$ for large θ_i and $\theta_i \equiv \sin \theta_i$ only for very small θ_i . The mean magnitude of $(\theta_\alpha + \theta_\beta + \theta_\gamma)$ obtained by the approximate method was 0.4% greater than that obtained using the exact solution when the central deflection $\delta = 0$ because of the cumulative effect of $\theta_i \equiv \sin \theta_i$ on each of the crack angles given the characteristic variation in α , β , and γ used in obtaining this estimate.

The approximate method of analysis served to demonstrate that the exact method was free from

gross errors. Both methods predicted a relationship between MOAs and CRAs broadly similar to that described by Holmgren & Norlin (2001).

3 DISCUSSION

3.1 Relationship between crack rotation angle and crack width

The analyses described above generated estimates of Crack Rotation Angles (CRAs) in each radial crack in a C1550 panel. In order to obtain estimates of the corresponding maximum crack widths it is necessary to select a relation between these two parameters. Since the analysis above was based on a rigid plate model of panel behaviour, it is consistent to also use a model incorporating rigid rotating plates at each hinge in which case the maximum crack width w at each hinge is simply related to the panel thickness t by the expression.

$$w = \theta t \quad (2)$$

This model assumes that each adjacent pair of panel sectors rotate about a line at the upper surface at which they meet. This model is quite satisfactory for plain concrete panels and quite possibly very lightly reinforced panels as well. However, most FRS displays a degree of post-crack residual strength that is manifested as a flexural-tensile stress distribution varying in magnitude through the lower portion of the C1550 panel. The tensile stresses are balanced by compressive stresses in the upper regions. There exists a neutral axis of zero stress at some depth from the upper surface which represents the point about which crack rotations should be calculated. Unfortunately, the indeterminate nature of the distribution of stress through a panel, plus the fact that membrane action causes a net tensile stress to act near the centre of the panel and net compressive stress to occur near the edges, means that it is very difficult to estimate the location of the neutral axis. Based on estimates of the rigidity of FRS before and after cracking (Bernard, 2008), the neutral axis is unlikely to be located further than $t/10$ from the upper surface, and thus the maximum crack width for most FRS will lie somewhere in the range $0.9\theta t < w < \theta t$.

Using the approximations presented above the distribution of crack widths occurring in C1550 panels can be found as a simple multiple of the distributions of CRAs developed in Section 2. The exact estimate of the crack width w in each radial crack for the symmetric cracking case (Figure 2) based on a rigid plate model is found as

$$w = \frac{\sqrt{3}\delta t}{r} \quad (3)$$

where r is the radius to the pivoted supports. Taking into account the distribution of MOAs (Eq. 1), which tends to increase the average crack width, and assuming an average neutral axis depth of $t/20$, a more realistic estimate of average maximum crack width in C1550 panels is

$$w = 0.95 \times 1.05 \times \frac{\sqrt{3}\delta t}{r} = \frac{1.73\delta t}{r} \quad (4)$$

3.2 Errors due to the rigid plate approximation

The analyses described above were all undertaken using a rigid plate model to estimate the magnitude of post-crack rotation angles in C1550 panels. This model ignores the fact that real panels suffer quasi-elastic deformation of the uncracked sectors when under load. A means of estimating the error in the CRAs due to the rigid plate approximation is therefore presented below.

Prior to cracking the CRAs are obviously zero, hence estimation of errors in the CRAs must commence at the point of cracking in the load-deflection curve. After initiation of the three radial cracks and attainment of the associated first peak in load resistance most FRC exhibits strain-softening behaviour, thus the maximum curvature experienced in the uncracked sectors of the panel will occur when the cracks are initiated. The fall in load resistance that usually occurs after cracking leads to elastic relaxation within the uncracked sectors and a reduction in associated curvature. The result is that the true CRAs at the three radial cracks will be smaller than suggested by the rigid plate model.

Laboratory measurements and elastic numerical modelling of structural behaviour (Figure 8, see also Bernard & Pircher, 2001) indicate that the deflection at the center of a C1550 panel in response to a point load is about 0.5 mm at the point of first cracking of the concrete matrix. Depending on the strength, age, and composition of the FRC, the stiffness at the center varies in the range 50–70 kN/mm. The total post-crack rotation θ_T experienced at one of the radial cracks can be approximated as

$$\theta_T = \theta_{RP} - \theta_e \quad (5)$$

where θ_{RP} is the rotation predicted by the rigid plate model and θ_e is the rotation due to quasi-elastic relaxation of the adjacent uncracked sectors. The magnitude of θ_e can be estimated by calculating the rigid plate rotation equivalent to a post-crack central deflection of 0.5 mm and scaling this in proportion to the fall in post-crack load resistance of the panel $P_{cr} - P_r$ divided by the load resistance at cracking P_{cr} , viz.

$$\theta_e = \frac{P_{cr} - P_r}{P_{cr}} \theta_{0.5} \quad (6)$$

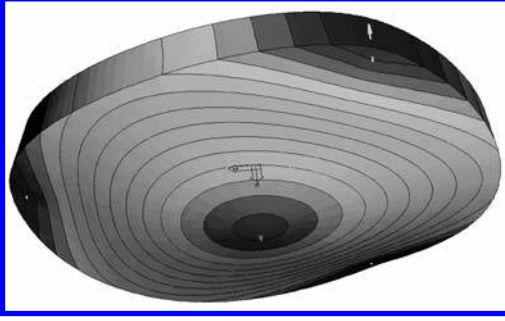


Figure 8. Distribution of normalized deflections on underside of a C1550 panel in response to a central point load.

The magnitude of $\theta_{0.5}$ is approximately equal to the rotation experienced at a post-crack central deflection of 0.5 mm for a FRC panel exhibiting no residual load resistance. This rotation has been taken to be modelled satisfactorily using the rigid plate approximation.

For a strain-softening FRC, errors in the CRAs due to the rigid plate approximation will be most significant immediately after cracking, but as the central deflection increases and the residual load resistance falls the relative magnitude of the errors become smaller. The significance of the error can be found by calculating the magnitude of θ_r and comparing this to θ_{RP} for a characteristic distribution of MOAs. The magnitude of error associated with the rigid plate approximation has been estimated for several load-deflection responses (Figure 9). These models represent successively more brittle FRC with a bi-linear fall in residual capacity immediately after cracking ranging from 99 to 25 percent of the peak load resistance. The results, shown in Figure 10, confirm that the errors in CRAs due to the rigid plate approximation are more significant the tougher the FRC is and the smaller the central deflection is. If the average post-crack capacity of commercial FRS is taken to be about 50% of peak load resistance at cracking, errors in the CRAs due to the rigid plate approximation will be no more than 1% for a deflection of about 5 mm or 5% for a deflection of 1 mm. For lightly reinforced FRC with a post-crack residual load capacity less than 25% of the peak the rigid plate approximation is sufficiently accurate at all deflections.

Note that the present geometric analysis based on rigid folding plates ignores the fact that the panel suffers membrane tension around the centre instead of pure flexure. The maximum crack widths experienced by a C1550 panel beyond ~20 mm central deflection are therefore somewhat greater than indicated by the analysis presented above.

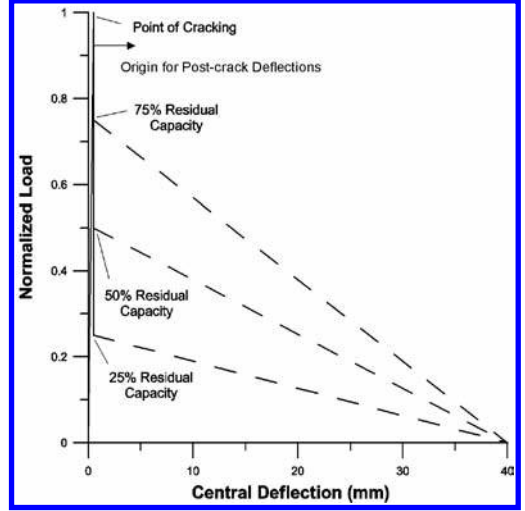


Figure 9. Simplified specimen behaviours used to assess magnitude of errors associated with the rigid plate approximation.

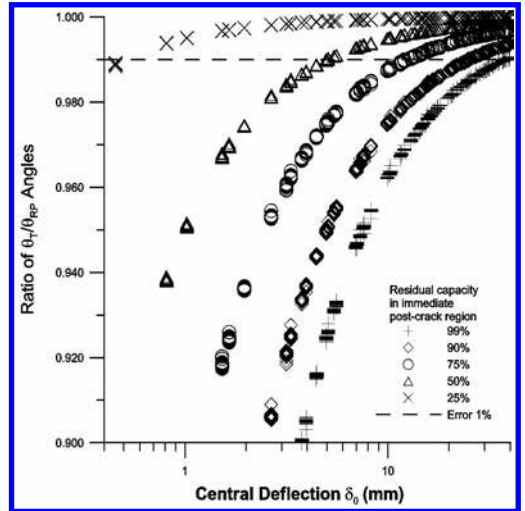


Figure 10. Magnitude of errors in average CRA involved in use of rigid plate approximation for models shown in Figure 9.

4 CONCLUSIONS

In this investigation an approximate closed-form solution for the angles of rotation in the three radial cracks that form in a C1550 panel was presented. This model predicts angles that are slightly larger than predicted using an exact rigid plate analysis developed elsewhere. Both models estimate average crack widths that are larger than predicted on the basis of the symmetric pattern of radial cracks

because average crack widths increase as the location of each crack approaches the pivoted supports in this test.

The rigid plate model was found to over-estimate crack widths compared to actual FRS panels that exhibit post-crack residual strength because of curvature in the uncracked portions of the panel and the fact that the neutral axis is typically not located at the upper surface of the specimen. However, membrane effects at larger deflections lead to a non-uniform distribution of crack width along each radial crack. At large deflections crack widths are wider near the centre and narrower near the edge of the specimen than is predicted by the rigid plate model.

REFERENCES

- ASTM International, Standard C1550. "Standard Test Method for Flexural Toughness of Fiber Reinforced Concrete (Using Centrally Loaded Round Panel)", ASTM, West Conshohocken, 2008.
- Banthia, N. & Trottier, J.-F., 1994. "Concrete Reinforced with Deformed Steel Fibers, Part 1: Bond-Slip Mechanisms", *ACI Materials Journal*, Vol. 91, No. 5, Sept-Oct, pp. 435–446.
- Bernard, E.S., 2008. "Post-Crack Flexural Modulus of Fibre-Reinforced Shotcrete", *Fifth International Symposium on Sprayed Concrete*, pp. 48–59. Lillehammer, Norway, April 21–24.
- Bernard, E.S. & Pircher, M., 2001. "The Influence of Thickness on Performance of Fiber-Reinforced Concrete in a Round Determinate Panel Test", *Cement, Concrete, and Aggregates*, CCAGDP, Vol. 23, No. 1, pp. 27–33.
- Bernard, E.S. & Xu, G.G., 2008. "The Effect of Radial Crack Locations on Load Resistance of C1550 Panels", *Journal of ASTM International*, Vol. 5, No. 10, Paper ID JA101739.
- Holmgren, J. & Norlin, B., 2001 "Properties and the use of the round determinately supported concrete panel for testing of fibre reinforced concrete", Workshop on *The Design of Steel Fibre Reinforced Concrete Structures*, Stockholm, June 12, The Nordic Concrete Federation.
- Johnson, N.L., Kotz, S. & Balakrishnan, N., 1994. *Continuous Univariate Distributions*, Vol. 1, 2nd Ed., John Wiley & Sons, New York.
- Kendall, M. & Stuart, A., 1979. *The Advanced Theory of Statistics, Volume 2: Inference and Relationship*, 4th Ed., Charles Griffin & Co., England.
- Kosa, K. & Naaman, A.E., 1990. "Corrosion of Steel Fiber Reinforced Concrete", *ACI Materials*, Vol. 87, No. 1, Jan-Feb, pp. 27–37.
- Law, A.M. & Kelton, W.D., 2000. *Simulation Modelling and Analysis*, 3rd Ed., McGraw-Hill, New York.
- Neville, A.M., 1995. *Properties of Concrete*, 4th Ed., Longman.
- Nordström, E., 2001. "Durability of Steel Fibre Reinforced Shotcrete with Regard to Corrosion", *Shotcrete: Engineering Developments*, Bernard (ed.), pp. 213–217, Swets & Zeitlinger, Lisse.
- Tran, V.G., Beasley, A.J. & Bernard, E.S., 2001. "The Application of Yield Line Theory to Round Determinate Panels", *Shotcrete: Engineering Developments*, Bernard (ed.), pp. 245–254, Swets & Zeitlinger, Lisse, 2001.
- Tran, V.N.G., Bernard, E.S., & Beasley, A.J., 2005. "Constitutive Modelling of Fiber Reinforced Shotcrete Panels", *Journal of Engineering Mechanics*, ASCE, Vol. 131, No. 5, May, pp. 512–521.

APPENDIX—APPROXIMATE DETERMINATION OF CRACK ROTATION ANGLES USING STRAIN COMPATIBILITY

For the purposes of the present analysis of a C1550 panel, the three sectors occurring between the radial cracks have been taken to be rigid plates, deflections are assumed to be small, and membrane tension is ignored. The model of the panel used for this analysis is shown in Figure A1. The three pivots are symmetrically arranged at the points 1, 2 and 3. The points A, B and C occur at the intersection between each radial crack and the edge of the panel. δ_α , δ_β , δ_γ and δ_0 are the deflections at the points A, B, C and the panel center O. The three midpoint offset angles (MOAs) α , β and γ define the location of the three radial cracks relative to the midpoints bisecting each sector and are measured clockwise positive. The rotations at the three radial cracks corresponding at these offset angles are θ_α , θ_β , and θ_γ . The radius of the panel is R and the radius to the supports is r . In this analysis it is assumed that $\theta_i \equiv \sin \theta_i$, hence errors will arise as deflections increase.

A1.1 Determination of deflections at edge of panel

The angle of crack rotation at each of the three radial cracks can be found by first calculating the deflections at the points where the radial cracks intersect the edge of the panel. Each of these deflections is found by a process based on rotation of the rigid plates on either side of each crack and the fact that the deflection at the supporting pivots is zero. The maximum deflection suffered by the panel is δ_0 at the centre. Taking a normal from one radial crack to each of the adjacent pivots (Figure A1), we have

$$x_1 = r \cos(\pi/3 + \beta) \quad \text{and} \quad x_2 = r \cos(\pi/3 - \beta) \quad (\text{A1})$$

$$h_1 = r \sin(\pi/3 + \beta) \quad \text{and} \quad h_2 = r \sin(\pi/3 - \beta) \quad (\text{A2})$$

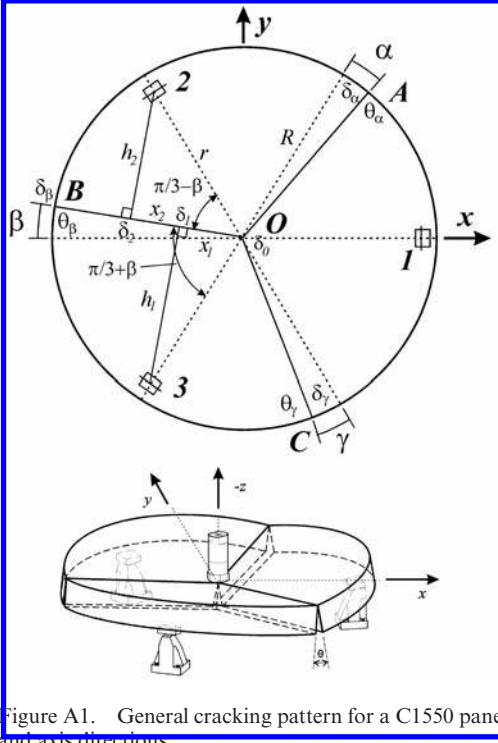


Figure A1. General cracking pattern for a C1550 panel and axis directions.

The deflections at the intersections between the normal from each pivot and the crack are

$$\delta_1 = \delta_0 + \frac{x_1}{R}(\delta_\beta - \delta_0) \quad \text{and} \quad \delta_2 = \delta_0 + \frac{x_2}{R}(\delta_\beta - \delta_0) \quad (\text{A3})$$

Given that the two sectors on each side of the crack are rigid plates, we have $\theta_\beta = \theta_1 + \theta_2$, thus

$$\theta_1 = \frac{\delta_1}{h_1} = \left[\delta_0 + \frac{r \cos(\pi/3 + \beta)}{R}(\delta_\beta - \delta_0) \right] / r \sin(\pi/3 + \beta) \quad (\text{A4})$$

$$\theta_2 = \frac{\delta_2}{h_2} = \left[\delta_0 + \frac{r \cos(\pi/3 - \beta)}{R}(\delta_\beta - \delta_0) \right] / r \sin(\pi/3 - \beta) \quad (\text{A5})$$

$$\theta_1 + \theta_2 = \left[\delta_0 + \frac{r \cos(\pi/3 + \beta)}{R}(\delta_\beta - \delta_0) \right] / r \sin(\pi/3 + \beta) + \left[\delta_0 + \frac{r \cos(\pi/3 - \beta)}{R}(\delta_\beta - \delta_0) \right] / r \sin(\pi/3 - \beta)$$

Collecting terms, we find for the deflection at β

$$\theta_\beta = \frac{\delta_0 \sin(\pi/3 - \beta) + r \cos(\pi/3 + \beta) \sin(\pi/3 - \beta) (\delta_\beta - \delta_0) / R}{r \sin(\pi/3 + \beta) \sin(\pi/3 - \beta)} + \frac{\delta_0 \sin(\pi/3 + \beta) + r \cos(\pi/3 - \beta) \sin(\pi/3 + \beta) (\delta_\beta - \delta_0) / R}{r \sin(\pi/3 + \beta) \sin(\pi/3 - \beta)} \quad (\text{A6})$$

The compound terms can be simplified using $\sin(\pi/3 + \beta) \sin(\pi/3 - \beta) = 2 \cos 2\beta + 1$, $\sin(\pi/3 + \beta) + \sin(\pi/3 - \beta) = \sqrt{3} \cos \beta$, and $\cos(\pi/3 + \beta) \sin(\pi/3 - \beta) + \cos(\pi/3 - \beta) \sin(\pi/3 + \beta) = \frac{\sqrt{3}}{2}$ thus

$$\theta_\beta (1 + 2 \cos 2\beta) \frac{R}{2\sqrt{3}} = \delta_0 \left[\frac{2R}{r} \cos \beta - 1 \right] + \delta_\beta \quad (\text{A7})$$

similarly

$$\theta_\alpha (1 + 2 \cos 2\alpha) \frac{R}{2\sqrt{3}} = \delta_0 \left[\frac{2R}{r} \cos \alpha - 1 \right] + \delta_\alpha \quad (\text{A8})$$

and

$$\theta_\gamma (1 + 2 \cos 2\gamma) \frac{R}{2\sqrt{3}} = \delta_0 \left[\frac{2R}{r} \cos \gamma - 1 \right] + \delta_\gamma \quad (\text{A9})$$

The relationship between the deflection at the extremity of each radial crack δ_i and the deflection in the adjacent cracks can be found using strain compatibility (Figure A2). Each sector between pairs of radial cracks experiences rigid body motion, hence the rotation about a line joining the pivot and the centre of the panel can be used to equate rotations associated with the deflections at the extremities of the panel.

Taking a normal from each crack to the line joining the pivot and the centre of the panel, we have

$$x_3 = R \cos(\pi/3 - \beta) \quad \text{and} \quad x_4 = R \cos(\pi/3 + \alpha) \\ h_3 = R \sin(\pi/3 - \beta) \quad \text{and} \quad h_4 = R \sin(\pi/3 + \alpha)$$

The deflection at the centre is δ_0 and the deflection at the pivot is zero. Hence, the deflection at the intersection between the normal from each crack (Figure A2) and the pivot line is found as

$$\delta_3 = \delta_0 \left[1 - \frac{R}{r} \cos(\pi/3 - \beta) \right] \quad \text{and} \quad \delta_4 = \delta_0 \left[1 - \frac{R}{r} \cos(\pi/3 + \alpha) \right] \\ \theta_3 = \frac{(\delta_\beta - \delta_3)}{h_3} = \left[\delta_\beta - \delta_0 \left[1 - \frac{R \cos(\pi/3 - \beta)}{r} \right] \right] / R \sin(\pi/3 - \beta) \\ \theta_4 = \frac{(\delta_\alpha - \delta_4)}{h_4} = \left[\delta_\alpha - \delta_0 \left[1 - \frac{R \cos(\pi/3 + \alpha)}{r} \right] - \delta_\beta \right] / R \sin(\pi/3 + \alpha) \quad (\text{A10})$$

Now $\theta_3 = \theta_4$, so

$$\delta_\alpha - \delta_0 \left[1 - \frac{R}{r} \cos(\pi/3 - \beta) \right] = \left[\delta_0 \left[1 - \frac{R}{r} \cos(\pi/3 + \alpha) \right] - \delta_\beta \right] \\ \times \frac{\sin(\pi/3 - \beta)}{\sin(\pi/3 + \alpha)} \quad (\text{A11})$$

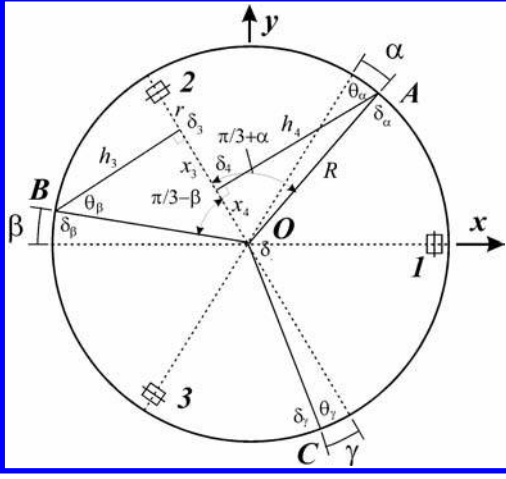


Figure A2. The effect of offset angles α and β on the deflections δ_α and δ_β in adjacent cracks.

Re-arranging terms, we find

$$\delta_\alpha + \delta_\beta \left[\frac{\sin(\pi/3 - \beta)}{\sin(\pi/3 + \alpha)} \right] = \delta_0 \left[1 + \frac{\sin(\pi/3 - \beta)}{\sin(\pi/3 + \alpha)} - \frac{R}{r} \times \left[\frac{\sin(2\pi/3 + \alpha - \beta)}{\sin(\pi/3 + \alpha)} \right] \right] \quad (\text{A12})$$

Similarly,

$$\delta_\beta + \delta_\gamma \left[\frac{\sin(\pi/3 - \gamma)}{\sin(\pi/3 + \beta)} \right] = \delta_0 \left[1 + \frac{\sin(\pi/3 - \gamma)}{\sin(\pi/3 + \beta)} - \frac{R}{r} \times \left[\frac{\sin(2\pi/3 + \beta - \gamma)}{\sin(\pi/3 + \beta)} \right] \right] \quad (\text{A13})$$

and

$$\delta_\gamma + \delta_\alpha \left[\frac{\sin(\pi/3 - \alpha)}{\sin(\pi/3 + \gamma)} \right] = \delta_0 \left[1 + \frac{\sin(\pi/3 - \alpha)}{\sin(\pi/3 + \gamma)} - \frac{R}{r} \times \left[\frac{\sin(2\pi/3 + \gamma - \alpha)}{\sin(\pi/3 + \gamma)} \right] \right] \quad (\text{A14})$$

A1.2 Solution of equations

The expressions listed above relating the deflection at the extremity of each radial crack represent three independent equations with three unknown deflections. This system of equations can be solved using

$$\begin{aligned} a\delta_\alpha + \delta_\gamma &= \delta_0 e \\ \delta_\alpha + b\delta_\beta &= \delta_0 f \\ \delta_\beta + c\delta_\gamma &= \delta_0 g \end{aligned} \quad \text{or}$$

$$\begin{bmatrix} a & 0 & 1 \\ 1 & b & 0 \\ 0 & 1 & c \end{bmatrix} \begin{Bmatrix} \delta_\alpha \\ \delta_\beta \\ \delta_\gamma \end{Bmatrix} = \delta_0 \begin{Bmatrix} e \\ f \\ g \end{Bmatrix} \quad (\text{A15})$$

which can be represented as

$$Ax = d$$

where

$$\begin{aligned} a &= \frac{\sin(\pi/3 - \alpha)}{\sin(\pi/3 + \gamma)} & b &= \frac{\sin(\pi/3 - \beta)}{\sin(\pi/3 + \alpha)} & c &= \frac{\sin(\pi/3 - \gamma)}{\sin(\pi/3 + \beta)} \\ e &= \left[1 + \frac{\sin(\pi/3 - \alpha)}{\sin(\pi/3 + \gamma)} - \frac{R}{r} \left[\frac{\sin(2\pi/3 - \alpha + \gamma)}{\sin(\pi/3 + \gamma)} \right] \right] \\ f &= \left[1 + \frac{\sin(\pi/3 - \beta)}{\sin(\pi/3 + \alpha)} - \frac{R}{r} \left[\frac{\sin(2\pi/3 + \alpha - \beta)}{\sin(\pi/3 + \alpha)} \right] \right] \\ g &= \left[1 + \frac{\sin(\pi/3 - \gamma)}{\sin(\pi/3 + \beta)} - \frac{R}{r} \left[\frac{\sin(2\pi/3 + \beta - \gamma)}{\sin(\pi/3 + \beta)} \right] \right] \end{aligned} \quad (\text{A16})$$

This system has the solution

$$x = A^{-1}d$$

where

$$A^{-1} = \frac{\text{adj}(A)}{\det(A)} = \frac{1}{abc + 1} \begin{bmatrix} bc & -1 & -b \\ a & ac & -1 \\ 1 & a & ab \end{bmatrix} \quad (\text{A17})$$

The solution for the three deflections is therefore obtained as

$$\begin{Bmatrix} \delta_\alpha \\ \delta_\beta \\ \delta_\gamma \end{Bmatrix} = \frac{\delta_0}{abc + 1} \begin{bmatrix} bc & -1 & -b \\ a & ac & -1 \\ 1 & a & ab \end{bmatrix} \begin{Bmatrix} e \\ f \\ g \end{Bmatrix} \quad (\text{A18})$$

or

$$\begin{aligned} \delta_\alpha &= \delta_0 \frac{ebc + f - gb}{abc + 1} & \delta_\beta &= \delta_0 \frac{acf + g - ce}{abc + 1} \\ \delta_\gamma &= \delta_0 \frac{abg + e - af}{abc + 1} \end{aligned} \quad (\text{A19})$$

These expressions can be substituted into Equations A7 to A9 to obtain the crack rotation angles.

Precision of the ASTM C1550 panel test and field variation in measured FRS performance

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ABSTRACT: The ASTM C1550 round panel test has now been used for Quality Control (QC) testing of Fibre Reinforced Shotcrete (FRS) for over 10 years. During that period a large number of specimens have been produced and tested and cumulatively represent a sizeable body of post-crack performance data with which to assess the statistics of FRS performance testing in the field. Inter-Laboratory Studies (ILS) have also been undertaken to determine the precision of the test method. The present paper describes two recent inter-laboratory studies to establish the single-operator and multi-laboratory precision of ASTM C1550. The studies indicated that coefficient of variation is an appropriate measure of the variability of test results. In general, the two studies resulted in similar indices of precision. In addition, numerous field data have been analyzed to investigate the underlying distribution of individual test results and the variation of test results among replicate test specimens.

1 INTRODUCTION

The ASTM C1550 round panel test, shown in Figure 1 (ASTM, 2008) was first developed in the late 1990s as an alternative to the EFNARC panel test (Roux et al., 1989; EFNARC, 1996), which is now known as the EN 14488 Part 5 square panel test (EN, 2006). Several aspects of fibre reinforced shotcrete (FRS) performance as revealed using the ASTM C1550 test method were first published by Bernard (2000). Later work compared the repeatability of test data using this method with that obtained using beams and EFNARC panels (Bernard, 2002). Additional studies have since examined other aspects of the use of round panels for quality control (QC) assessment in the field (Hanke et al. 2002) and possible sources of error in this test method (Bernard & Pircher, 2001; Bernard, 2005; Bernard & Xu, 2008). As a result, the ASTM C1550 round panel test is a relatively well understood test method and results obtained using this test are specified in guides on the use of shotcrete (ACI, 2008; CIA, 2008).

The ASTM C1550 test offers many advantages over alternative methods of performance assessment, including lower production and testing costs and superior within-batch repeatability. It is now used as part of QC programmes for most projects employing shotcrete in Australia, with the result that approximately 5000 specimens are pro-

duced and tested annually. The fact that many of these specimens have been tested using the same servo-controlled testing machine at TSE P/L in Sydney has made it possible to accumulate a large body of data concerning within-batch variability in measured FRS performance for specimens produced in the field. These data have been analysed below to identify characteristic levels of variability and information about the distribution of within-batch results so that the test method can be used rationally and for maximum benefit in future projects.

The present study examines two related topics: the results of a pair of inter-laboratory studies

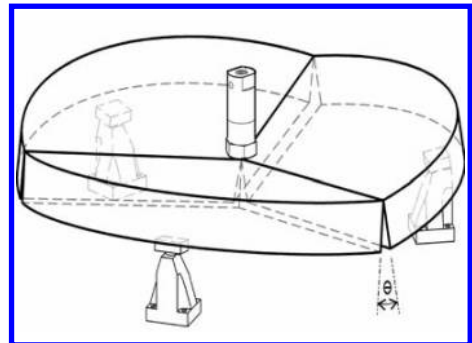


Figure 1. ASTM C1550 round panel test.

(ILS) involving laboratories in Australia, USA, England, and Sweden; and the characteristic distribution of within-batch variability for pairs of specimens produced as part of field QC programmes in Australia.

2 INTER-LABORATORY STUDIES

2.1 Descriptions of studies

The first inter-laboratory study (ILS) involved the production of four sets of round panel specimens using two fibre-reinforced concrete (FRC) mixtures and two FRS mixtures. The fibre type and content and the target compressive strength of each mixture are given in Table 1. Forty (40) panels were made from each mixture and were tested subsequently using four different servo-hydraulic testing machines located at TSE P/L and the University of Western Sydney (UWS), Australia. Table 2 indicates the testing machines and the measured load train stiffness (k). Ten replicate specimens from each mixture were tested using each machine.

Specimens were made from the FRC or FRS mixture by casting in moulds. Concrete was supplied from a dry-batch ready-mixed plant and delivered as a 3.0 m³ batch in a 5.6 m³ truck-mounted agitator. Fibres were added on top of the concrete and mixed in for 15 minutes at high-speed rotation. The round moulds were laid horizontally on the ground and concrete with fibres mixed in was dispensed via the chute directly into the moulds. The concrete was vibrated using an internal vibrator, then screeded and floated using the procedure described in the Appendix of ASTM C1550.

After finishing, the specimens were covered with plastic sheeting and allowed to harden for two days before stripping and transfer to tanks containing

lime-saturated water. Immersion curing in water continued for 3–5 months after which the specimens were withdrawn and tested in a surface-moist condition. The 10 specimens tested using each of the machines listed in Table 2 were randomly selected from the 40 produced as part of each casting trial.

Because the first ILS involved only four testing machines from two facilities located in Sydney, the results were regarded as unlikely to satisfy the requirements for a general estimation of multi-laboratory precision of ASTM C1550. A second larger study was therefore undertaken with support from the ASTM ILS program.

The second inter-laboratory study involved six sets of round panel specimens using three FRC mixtures and three FRS mixtures. The fibre type and content and the target compressive strength of each mixture are given in Table 3. Thirty-six (36) panels were made from each mixture and were tested subsequently using six different servo-hydraulic testing machines located at five facilities (TSE P/L; UWS, BASF in Cleveland, Ohio; University of Greenwich, Kent, U.K.; and University of Luleå, Sweden). Table 4 indicates the testing machines and the measured load train stiffness (k). The results from each machine at UWS were treated as coming from a different “laboratory”. Six replicate specimens from each mixture were tested using each machine.

Production of the specimens was achieved either by casting or spraying. Concrete was supplied from a dry-batch ready-mixed plant and delivered

Table 1. Mixtures used in first ILS.

Set	Fibre	Description
A1	9 kg/m ³ Strux 85/50	25 MPa cast FRS
A2	60 kg/m ³ Wavecut 25 × 0.7 mm	25 MPa cast FRC
A3	30 kg/m ³ Novotex 0730	32 MPa cast FRC
A4	8 kg/m ³ Kyodo 48 mm	32 MPa cast FRS

Table 2. Testing machines used in first ILS.

Rig	Facility	Machine	k (N/mm)
1	UWS	3000 kN Instron 8506	72300
2	UWS	1000 kN Instron 8800	72400
3	TSE	100 kN MTS 244.21	51300
4	TSE	250 kN MTS 244.31	53100

Table 3. Mixtures used in second ILS.

Set	Fibre	Description
B1	6.4 kg/m ³ Kyodo 48 mm	40 MPa FRS
B2	8 kg/m ³ Enduro 50 mm	32 MPa FRS
B3	40 kg/m ³ Dramix RC 65/35	32 MPa FRS
B4	37.5 kg/m ³ Novotex 0730	20 MPa cast FRC
B5	5 kg/m ³ Shogun 48 mm	20 MPa cast FRC
B6	25 kg/m ³ Novotex 1050	25 MPa cast FRC

Table 4. Testing machines used in second ILS.

Facility	Machine	k (N/mm)
UWS	3000 kN Instron 8506	72300
UWS	1000 kN Instron 8804	72400
TSE	250 kN MTS 244.31	53100
BASF	1300 kN Instron 300	—
Greenwich	DARTEC 500 kN	120900
Luleå	Eland 1000 kN	157000

as a 3.0 m³ batch in a 5.6 m³ truck-mounted agitator. Fibres were added on top of the concrete and mixed in for 15 minutes on high speed rotation. For the cast specimens, round moulds were laid horizontally on the ground and concrete with fibres mixed in was dispensed via the chute directly into the moulds. The concrete was vibrated using an internal vibrator, then screeded and floated. The sprayed specimens were produced by placing the moulds against a rack at approximately 45° to the horizontal and manually spraying shotcrete into the forms starting at the bottom and moving upward. Set accelerator was used at a dosage rate of approximately 5% by mass of cement. The inclined specimens were screeded and floated in the inclined position before being lowered to the horizontal position and finished with a steel float.

After finishing all specimens were covered with plastic sheeting and allowed to harden for two days before stripping and transfer to tanks containing lime-saturated water. Immersion curing in water continued for 6 months after which the specimens were stored in an external shaded but dry environment for 18 months. They were then placed in crates for shipping to the various facilities. All specimens were tested within a period of approximately 4 weeks and at an age of approximately two and a half years. The specimens to be tested at each facility were selected successively from the 36 panels produced for each mixture. Thus, every sixth panel was tested on the same machine (e.g., panels 1, 7, 13, etc.). This ensured that the six specimens from each mixture to be tested on the same machine represented material from throughout the entire load of concrete used to make the specimens.

2.2 Data analysis

In both inter-laboratory studies the measured parameters included peak load and energy absorption sustained at 5, 10, 20, and 40 mm central deflection. The Appendix provides test results from the first ILS and the results from the second ILS are available in Bernard & Carino (2009).

ASTM C1550 requires the formation of at least three radial cracks in a panel specimen for a valid test. In each ILS, some specimens did not meet this requirement, so some test results had to be discarded. Therefore, the data sets were not “balanced”, that is, they did not have the same number of replicate test results for each combination of laboratory and mixture (or “cell”). This affected the data analysis methods that were used.

The objective of the data analysis in an ILS programme is to determine the single-operator index of precision (standard deviation or coefficient of variation [COV]) and the multi-laboratory index of precision. The former is usually called “repeatability” and the latter is usually called “reproducibility”.

For balanced data sets, that is, equal number of replicates in each cell, procedures for calculating these quantities can be found in ASTM C802 or ASTM E691.

In the first ILS, only one specimen failed to meet the three radial-cracks requirement of ASTM C1550. The single-operator variance (standard deviation squared) for each mixture was obtained by pooling the variances from the four testing machines. The between laboratory component of variance (s_b^2) for each mixture was obtained by using analysis of variance (ANOVA) with a correction factor for unequal data sets (Mandel & Paule 1970; Montgomery 1984). The following equation was used:

$$s_b^2 = \frac{(p-1)MS_b - MS_w}{\sum n_i - \sum \frac{n_i^2}{n_i}} \quad (1)$$

In Eqn. (1), MS_w is the within-laboratory mean square and MS_b is the between-laboratory mean square obtained from the ANOVA table. The number of laboratories is p and the number of replicates for each laboratory is n_i .

In the second ILS, 11 specimens had less than three radial cracks: mixture 4 had two cases; mixture 5 had three cases; and mixture 6 had six cases. Although the between-laboratory component of variance can be estimated using Eqn. (1), the method described by Mandel & Paule (1970, 1982) was instead used. Mandel & Paule argue that the analysis of variance approach gives equal weighting to each result, and a better approach is to use a weighting based on the inverse of the variance associated with each cell (particular combination of laboratory and mixture).

Basically, the Mandel-Paule method uses an iterative procedure to obtain an estimate of the between-laboratory component of variance (s_b^2) and of the weighted grand mean for each test parameter (peak load and energy absorption). The steps used to estimate the between-laboratory component of variance and the grand mean for each parameter in the second ILS are presented in Bernard & Carino (2009).

In general, the between-laboratory component of variance is defined as follows:

$$s_b^2 = s_Y^2 - \frac{s_w^2}{n} \quad (2)$$

In Eq. (2), s_Y^2 is the variance of the laboratory averages for a particular mixture, s_w^2 is the pooled single-operator variance, and n is the number of replicates. Where there is a small variation among the laboratory averages, Eqn. (2) may result in a

negative value for s_b^2 . In such cases, ASTM C802 and ASTM E691 state that s_b^2 is to be assigned a value of zero.

ASTM C1550 defines a test result as the average of at least two individual determinations. For the case where two replicate determinations comprise a test result, the multi-laboratory variance (s_R^2) is computed as follows:

$$s_R^2 = s_b^2 + \frac{s_w^2}{2} \quad (3)$$

The multi-laboratory variance for individual determinations would be obtained by replacing the number 2 in Eq. (3) with the number 1.

2.3 Results of data analysis

2.3.1 Single-operator precision

The values of the pooled single-operator standard deviations s_w for peak load are shown as a function of the average peak load for each mixture in Figure 2.

Several observations are noted. First, the range of the average peak loads is limited. Second, the standard deviations from the first ILS appear to have a linear dependence on average load, which implies that the coefficient of variation may be used as the measure of variability. Third, for the second ILS, the standard deviation did not appear to depend on the peak load. The last observation implies that the standard deviation is constant, which is not typical of concrete strength tests. It was found that if straight lines are fitted through the origin for each data set, the slopes are the same with a value of 0.062. Thus it may be concluded that the single-operator coefficient of variation for

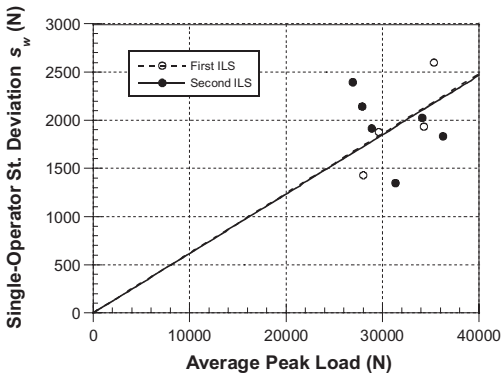


Figure 2. Single-operator standard deviation for peak load as a function of average peak load for each mixture.

peak load is 6.2% with a 95% confidence interval of 5.2% to 7.2%.

The single-operator standard deviation for the energy absorption parameters is shown as a function of the average value in Figure 3. There are three significant observations. First, the standard deviation appears to increase linearly with average energy absorption. Second, the results for the two ILS programs appear to be similar. Third, the results for the different energy parameters appear to fall along a common line.

The first observation supports the use of coefficient of variation as the measure of single-operator variability. To examine whether the results for each energy parameter from the two ILS programs can be combined, the confidence intervals for the slopes of the best-fit straight lines through the origin for each ILS were compared. Columns 4 and 5 in Table 5 show the 95% confidence limits

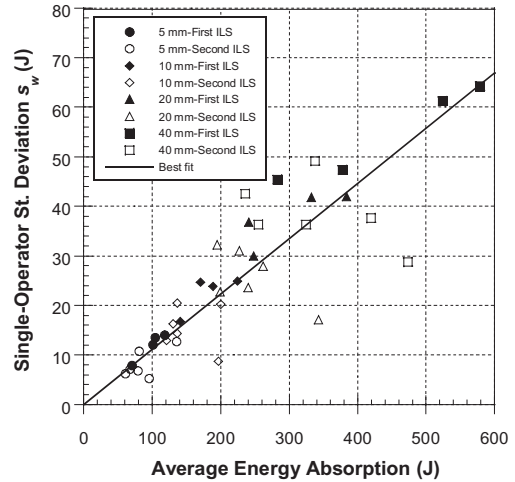


Figure 3. Single-operator standard deviation for energy absorption as a function of average energy absorption for each mixture.

Table 5. Single-operator coefficient of variation COV_w for energy absorption parameters.

Energy	ILS	COV_w	Lower 95%	Upper 95%	Combine	COV_w
T-5	1	12.1%	11.0%	13.2%	Yes	10.5%
	2	9.2%	6.6%	11.9%		
T-10	1	12.3%	10.0%	14.7%	Yes	10.9%
	2	9.7%	5.7%	13.6%		
T-20	1	12.3%	9.6%	14.9%	Yes	11.1%
	2	9.8%	5.5%	14.2%		
T-40	1	12.0%	9.4%	14.6%	Yes	11.3%
	2	10.4%	6.0%	14.8%		

for the best-fit slopes expressed in percent (these represent the single-operator coefficients of variation). If the confidence intervals of the coefficients of variation for each energy parameter overlap, the two sets of ILS data can be combined. The column labeled “Combine” indicates whether the confidence intervals of COV_w overlap. It is seen that for each energy parameter, the data from the two ILS programs can be combined. The last column of Table 5 shows the resulting best-fit value of COV for each energy parameter. These values are remarkably similar, and it is reasonable to compute a single best-fit slope for the combined data in Fig. 3. The computed slope for a line through the origin is 0.111 with a 95% confidence interval of 0.101 to 0.121. Thus, for the energy absorption parameters, the single-operator COV for the ASTM C1550 test is approximately 11%.

2.3.2 Multi-laboratory precision

The multi-laboratory standard deviation s_R was calculated by taking the square root of the variance computed in accordance with Eq. (3). Figure 4 shows the resulting values of s_R for peak load for the two ILSs. There are two notable observations: firstly, the standard deviation appears to increase with average peak load; and secondly, the linear trends do not appear to be the same for the two studies. Thus it can be concluded that COV is an appropriate measure of multi-laboratory precision. For the first ILS, the best-fit value for multi-laboratory coefficient of variation (COV_R) is 5.3% with a 95% confidence interval of 4.1% to 6.5%. While for the second ILS, the best-fit value is 8.9% with a 95% confidence interval of 7.4% to 10.3%. Because the confidence intervals do not overlap, the COV_R values are different for these two studies. Thus an upper estimate of multi-laboratory COV for peak load of the ASTM C1550 test is about 9%.

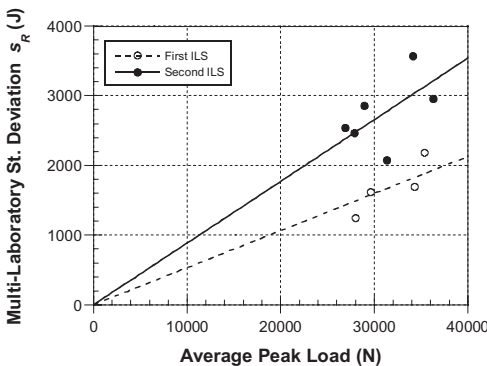


Figure 4. Multi-laboratory standard deviation for peak load as a function of average peak load for each mixture.

The multi-laboratory standard deviation for the energy absorption parameters is shown as a function of the average value in Figure 5. It is clear that the standard deviation increases with the average value of energy absorbed. The same approach as used for the single-operator results was used to examine whether the multi-laboratory results from the two studies can be combined, and whether the results for the different energy parameters can be combined.

Table 6 shows the 95% confidence intervals for the best-fit values of COV_R for the various energy absorption parameters from the two ILS programs. The overlapping confidence intervals for each parameter indicate that the results from the two ILSs can be combined. The resulting best-fit values for COV_R for each energy parameter using the combined data are shown in the last column of

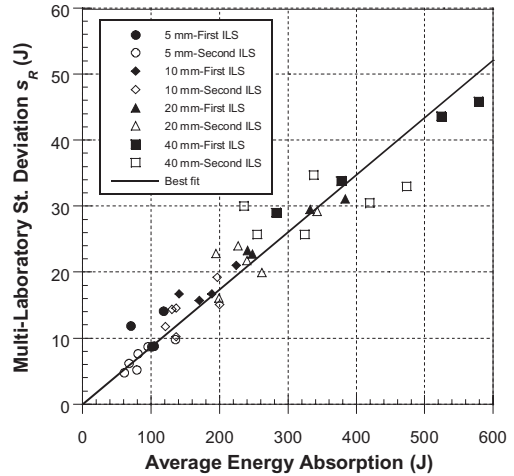


Figure 5. Multi-laboratory standard deviation for energy absorption as a function of average energy absorption for each mixture.

Table 6. Multi-laboratory coefficient of variation COV_R for energy absorption parameters.

Energy	ILS	COV_R	Lower 95%	Upper 95%	Combine	COV_R
T-5	1	10.8%	5.7%	15.9%	Yes	9.3%
	2	8.0%	6.8%	9.3%		
T-10	1	9.6%	7.8%	11.4%	Yes	9.3%
	2	9.1%	7.6%	10.7%		
T-20	1	8.7%	7.7%	9.8%	Yes	8.9%
	2	9.0%	7.6%	10.4%		
T-40	1	8.5%	7.2%	9.7%	Yes	8.4%
	2	8.4%	6.3%	10.4%		

Table 6. The COV_R values for each energy parameter are similar to each other, and it is appropriate to combine all the data and compute a single value. When this is done, the best-fit value of COV_R for the energy absorption parameters is 8.7% with a 95% confidence interval of 8.3% to 9.1%. Thus for the energy absorption parameters, the multi-laboratory COV for the ASTM C1550 test is approximately 8.7%. Keep in mind that this is based on comparing the averages of the results of *two* individual test specimens (Eqn. (3)).

2.4 Summary

The data from two ILS programs of ASTM Test Method C1550 have been analyzed to provide information on the inherent precision of the method. The first ILS involved four materials, four testing machines and 10 replicates. The second ILS involved six materials, six testing machines, and six replicates.

Because some of the individual test results had to be discarded, the methods in ASTM C802 and ASTM E691 for evaluating the between-laboratory component of variance were not applicable. For the first ILS, only one test result for each performance parameter had to be discarded (see the Appendix for all results obtained in the first ILS). In this case, analysis of variance was used to compute the between-laboratory component of variance. For the second ILS, 11 tests results had to be discarded, and the method of Mandel & Paule (1970, 1982) was used. An ASTM Research Report giving the details of the data analysis of the second ILS is available (Bernard & Carino 2009).

ASTM C1550 requires that a test result includes at least two individual determinations. Therefore, the multi-laboratory variance s_R^2 was computed for a test result defined as the average of two individual determinations (Eq. (3)).

The analysis showed that, in general, the single-operator and multi-laboratory standard deviations increased with the average value of the property. Therefore, the COV was used as the measure of precision. Linear regression for lines passing through the origin was used to obtain the best-fit estimates for the single-operator and multi-laboratory coefficients of variation. The 95% confidence intervals for the estimates were used to assess whether the results from the two ILS programmes could be combined. For the energy absorption parameters, it was found that the level of deflection did not have a significant effect on the variability of measured energy absorption.

Analysis of the data obtained from the two ILS programmes is summarized in Table 7. For the multi-laboratory COV for the peak load the two ILS programs gave different values. The COV_w of 9% is

Table 7. Single-operator and multi-laboratory coefficients of variation for ASTM C1550.

Performance parameter	Single-operator, COV_w	Multi-laboratory, COV_R
Peak load	6.2%	≈9%
Energy absorption	11%	8.7%

the higher value obtained from the second ILS and is an upper estimate. In accordance with ASTM C670 or ASTM E177, the maximum difference (as percent of their average) between two test results that is expected in 95 out of 100 cases is obtained by multiplying these coefficients of variation by 2.8. Again, for the multi-laboratory case, a “result” is the average of *two* individual determinations.

3 VARIANCE IN FIELD-PRODUCED DATA

The ILS data presented above were derived from specimens produced under laboratory conditions. The objective of the second study presented in this paper was to assess the within-batch variability of performance parameters obtained using ASTM C1550 panels made in the field. This study also examined the characteristic distribution of individual results within a set of QC specimens and the distribution of within-batch COV data.

3.1 Distribution of within-batch results

Over the last 10 years the ASTM C1550 panel test has been used for QC purposes on numerous tunneling projects and in most underground metalliferous mines in Australia. The 100 kN MTS testing machine operated by TSE P/L has been used to conduct at least 6,000 ASTM C1550 tests on specimens produced in the field over this period. Several projects have involved in excess of 1000 panels, usually produced in sets of 2 or 3. The individual specimen results within each set of QC results were normalized with respect to the mean for each corresponding set and plotted as a histogram in Figure 6. This data therefore represents the distribution of the relative magnitude of individual test results within each batch with respect to the arithmetic mean for that batch.

The frequency distribution shown in Figure 6 shows that a modified Gaussian curve fitted to the field data using singular value decomposition (Johnson et al. 1994) produced an exceptionally good fit with $r^2 = 0.9960$. This function is plotted as the solid line in Figure 6. The expression for the Modified Gaussian curve is

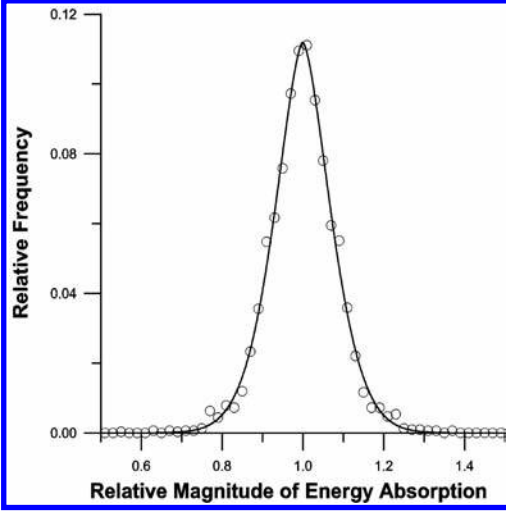


Figure 6. Distribution of the magnitude of energy absorption data at 40 mm relative to the within-batch mean for the corresponding batch (circles) for 3250 ASTM C1550 panels produced as part of QC programmes in Australian mines and tunnels.

$$y = a \exp \left[-\frac{1}{2} \left(\frac{|x-b|}{c} \right)^d \right] \quad (4)$$

in which $a = 0.111909$, $b = 0.99985$, $c = 0.064243075$, and $d = 1.602313$. When the conventional Gaussian curve is used (for which $d = 2$), $r^2 = 0.993$. These results indicate that individual energy absorption data from field-produced specimens are normally distributed about the arithmetic mean.

3.2 Within-batch COV

The within-batch COV was calculated for energy absorption at 40 mm central deflection for all pairs of field-produced specimens available and has been presented as a frequency distribution in Figure 7. The distribution shows that the within-batch COV is asymmetrically distributed with a long tail in the distribution toward larger values. This shows that for a sample size of two there is large variability in the calculated coefficient of variation (or standard deviation).

The overall shape of the frequency distribution defined by the points in Figure 7 is similar to Helmert's equation (Deming, 1950) for the sampling distribution function of the sample standard deviation computed from two replicate specimens taken from a normal distribution. Thus the data were fitted with a function analogous to Helmert's

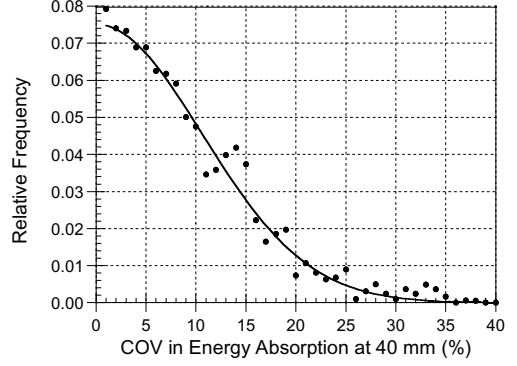


Figure 7. Distribution of within-batch coefficients of variation (COV) in energy absorption at 40 mm for replicate specimens ($n = 2$) produced in selected mining and tunneling projects within Australia.

distribution with the COV substituted for the standard deviation.

For the case where standard deviation is based on $(n - 1)$ degrees of freedom, Helmert's distribution function for the standard deviation is given by the following:

$$f(s) = 2 \frac{\left(\frac{n-1}{2\sigma^2} \right)^{(n-1)/2}}{\Gamma((n-1)/2)} s^{n-2} e^{-(n-1)s^2/(2\sigma^2)} \quad (5)$$

in which Γ is the gamma function. In Eqn. (5), σ is the population standard deviation and s is the computed value of standard deviation based on n replicate tests. For the case $n = 2$, $\Gamma(0.5) = 1.7725$. If s is replaced with V (for coefficient of variation), the distribution function for the coefficient of variation based on two replicates is as follows

$$f(V) = \frac{0.7979}{V_{\text{inf}}} e^{-0.5V^2/V_{\text{inf}}^2} \quad (6)$$

The only adjustable parameter in Eq. (6) is the value of V_{inf} , which is the true coefficient of variation for the population. Figure 7 shows the resulting best-fit curve to the data, and the estimated value of V_{inf} is 10.6%.

The average within-batch COV for the data of energy absorption at 40 mm central deflection obtained from field-produced panels is about 9% (a similar level was obtained for energy absorbed at 5, 10, and 20 mm central deflection). However from the fitted distribution function, the within-batch COV for the population is estimated to be 10.6%. This difference may be attributed to the effect of the number of replicates on the computed COV, as discussed in the authors' companion paper in

these proceedings (Bernard et al., 2010). The COV obtained using two replicate specimens will underestimate the true COV for the population. From the two ILS programmes, the within-batch COV for energy absorption parameters was estimated to be 11%. This compares favorably with the estimate based on field results.

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REFERENCES

- American Concrete Institute 506.1R, 2008. *Guide to Fiber-Reinforced Shotcrete*, ACI, Farmington Hills.
- ASTM International, 2008. Practice C670, "Standard Practice for Preparing Precision and Bias Statements for Test Methods for Construction Materials."
- ASTM International, 2008. Practice C802, "Standard Practice for Conducting an Interlaboratory Test Program to Determine the Precision of Test Methods for Construction Materials," ASTM, West Conshohocken, Pa.
- ASTM International, 2008. Test Method C1550, "Standard Test Method for Flexural Toughness of Fiber Reinforced Concrete (Using Centrally Loaded Round Panel)," ASTM, West Conshohocken, Pa.
- ASTM International, 2008. Practice E177, "Standard Practice for Use of the Terms Precision and Bias in ASTM Test Methods," ASTM, West Conshohocken, Pa.
- ASTM International, 2008. Practice E691, "Standard Practice for Conducting an Interlaboratory Study to Determine the Precision of a Test Method," ASTM, West Conshohocken, Pa.
- Bernard, E.S., 2000. "Behaviour of round steel fibre reinforced concrete panels under point loads", *Materials and Structures*, RILEM, Vol. 33, April, pp. 181–188.
- Bernard, E.S. 2002. "Correlations in the behavior of fiber reinforced shotcrete beam and panel specimens", *Materials and Structures*, RILEM, Vol. 35, April, pp. 156–164.
- Bernard, E.S., 2005. "The Role of Friction in Post-crack Energy Absorption of Fiber Reinforced Concrete in the Round Panel Test", *Journal of ASTM International*, January, Vol. 2, No.1, pp. 1–12.
- Bernard, E.S. and Pircher, M., 2001. "The Influence of Thickness on Performance of Fiber-Reinforced Concrete in a Round Determinate Panel Test", *Cement, Concrete, and Aggregates*, CCAGDP, Vol. 23 No. 1, pp. 27–33.
- Bernard and Carino, N., 2009, "Interlaboratory Study to Establish Precision Statements for ASTM C1550, Standard Test Method for Flexural Toughness of Fiber Reinforced Concrete (Using Centrally Loaded Round Panel)", ASTM Report RR-XXXX, ASTM International, www.astm.org.
- Bernard, E.S., Xu., G.G. and Carino, N.J., 2010. "Influence of the number of specimens in a batch on apparent variability in FRS performance assessed using ASTM C1550 panels," *Shotcrete: Elements of a System*, Taylor & Francis, London.
- CIA, 2008. *Shotcreting in Australia: Recommended Practice*, Concrete Institute of Australia & AuSS, Sydney.
- Deming, W.E., 1950. *Some Theory of Sampling*, Wiley.
- EFNARC, 1996. *European Specification for Sprayed Concrete*, European Federation of National Associations of Specialist Contractors and Material Suppliers for the Construction Industry.
- EN 14488 *Testing Sprayed Concrete*, European Standards (Euronorm), European Committee for Standardisation.
- Hanke, S.A., Collis, A. and Bernard, E.S. 2001. "The M5 motorway tunnel: an education in Quality Assurance for fiber reinforced shotcrete", *Shotcrete: Engineering Developments*, Bernard (ed.), 145–156, Swets & Zeitlinger, Lisse.
- Johnson, N. L., Kotz, S. and Balakrishnan, N. 1994. *Continuous Univariate Distributions* Vol. 1, 2nd Ed., John Wiley & Sons, New York.
- Kendall, M. and Stuart, A. 1979. *The Advanced Theory of Statistics, Volume 2: Inference and Relationship*, 4th Ed., Charles Griffin & Co., England.
- Law, A.M. and Kelton, W.D., 2000. *Simulation Modeling and Analysis*, 3rd Ed., McGraw-Hill New York.
- Mandel, J. and Paule, R. C., 1970. "Interlaboratory Evaluation of a Material with Unequal Number of Replicates", *Analytical Chemistry*, Vol. 42, No, 11, pp. 1194–1197.
- Montgomery, D.C., 1984. *Design and Analysis of Experiments*, John Wiley & Sons Inc.
- Paule, R.C. and Mandel, J., 1982. "Consensus Values and Weighting Factors", *Journal of Research of the National Bureau of Standards*, Vol. 87, No. 5, pp. 377–385.
- Roux, J., Cheminais, J., Rivallain, G. and Mourand, C., 1989. 'Béton projeté par voie sèche avec incorporation de fibres', *Tunnels et Ouvrages Souterrains*, 92 pp. 61–97.

APPENDIX

This appendix contains summarized results for all four sets of specimens produced and tested as part of the first ILS reported in the main text of this paper.

Table A1. Test results from first ILS, Specimen Set 1.

Machine	Peak load (N)	Energy absorption			
		5 mm (J)	10 mm (J)	20 mm (J)	40 mm (J)
Instron 8506 3000 kN	31791	103	196	355	574
	31159	112	208	367	582
	31174	129	252	451	717
	30159	95	174	298	470
	32157	113	210	368	573
	33050	112	203	350	547
	29998	99	185	324	501
	28578	95	178	316	505
	27280	79	147	260	418
	31120	120	230	406	641
	26471	76	143	255	416
	30733	110	206	362	539
Instron 8800 1000 kN	27597	99	185	322	508
	31218	97	177	310	498
	31064	114	214	373	588
	30242	96	179	312	496
	29736	93	173	301	476
	27531	99	188	340	554
	31153	95	176	308	481
	33027	125	231	396	582
	28087	90	169	302	490
	33104	122	229	400	625
MTS 243 100 kN	31729	127	240	416	607
	28853	85	158	279	444
	27049	92	175	310	502
	29597	99	182	316	499
	31357	100	183	323	516
	31843	102	183	309	454
	29637	106	197	343	543
	26888	98	184	328	528
	27025	87	165	291	472
	28101	95	183	323	519
MTS 244 250 kN	27858	101	194	347	543
	27031	95	181	320	516
	28694	91	173	304	480
	27307	101	192	336	538
	32035	103	191	344	565
	27375	94	179	321	511
	26707	90	163	280	442
	28341	98	183	315	489

Table A2. Test results from first ILS, Specimen Set 2.

Machine	Peak load (N)	Energy absorption			
		5 mm (J)	10 mm (J)	20 mm (J)	40 mm (J)
Instron 8506 3000 kN	37036	119	189	243	244
	38949	113	182	253	292
	37947	112	184	261	305
	39415	128	214	303	347
	25170	89	144	201	237
	36977	111	178	250	296
	33456	98	162	233	278
	33805	96	158	229	276
	37335	109	183	255	299
	35472	81	126	174	205
	35281	99	163	233	276
	37053	108	175	249	294
Instron 8800 1000 kN	39279	121	199	291	362
	35281	109	181	267	332
	37458	120	201	290	342
	32737	78	125	173	202
	38761	112	187	271	318
	35077	87	137	197	238
	35746	105	165	230	268
	35268	93	145	203	240
	33244	88	140	194	224
	39011	119	190	259	290
MTS 243 100 kN	34754	110	173	234	263
	37222	97	159	226	269
	37670	111	181	257	306
	39963	101	158	224	266
	35787	110	174	243	284
	34190	111	184	265	314
	36970	124	207	299	361
	35084	94	148	205	219
	33036	108	186	271	320
	32464	104	174	245	278
MTS 244 250 kN	32583	109	178	252	299
	35414	131	223	324	383
	33674	95	157	223	266
	32267	89	142	201	241
	33139	113	189	270	324
	32416	91	147	210	242
	35186	99	164	233	272
	32605	80	132	189	233

Table A3. Test results from first ILS, Specimen Set 3.

Machine	Peak load (N)	Energy absorption			
		5 mm (J)	10 mm (J)	20 mm (J)	40 mm (J)
Instron 8506 3000 kN	30513	100	184	311	473
	32286	119	221	374	550
	35177	123	228	390	590
	36038	141	255	423	636
	38282	144	270	460	701
	37321	156	287	481	717
	36041	121	227	391	590
	33437	121	234	394	590
	33856	136	250	414	617
Instron 8800 1000 kN	33252	108	201	311	442
	36294	121	219	362	530
	31850	100	181	306	465
	34153	120	228	397	617
	36056	139	263	445	673
	34519	121	227	395	605
	38351	138	254	434	643
	32821	149	269	444	655
	33377	127	236	416	635
MTS 243 100 kN	34886	128	235	394	588
	33016	98	191	331	504
	34231	104	194	327	479
	33760	100	198	349	526
	39245	145	269	435	604
	33751	104	203	351	532
	37592	127	247	422	635
	34283	114	213	364	545
	36647	128	248	426	640
MTS 244 250 kN	36112	123	238	415	635
	34395	102	199	341	498
	33195	104	210	376	579
	32620	109	208	363	548
	32768	107	212	372	576
	34544	107	204	355	534
	34090	114	222	391	604
	33193	101	214	381	587
	32042	99	200	364	578
	31315	105	202	355	545
	31708	110	217	378	578
	30991	89	184	330	509

Table A4. Test results from first ILS, Specimen Set 4.

Machine	Peak load (N)	Energy absorption			
		5 mm (J)	10 mm (J)	20 mm (J)	40 mm (J)
Instron 8506 3000 kN	26624	68	132	228	353
	28214	73	144	254	394
	24408	89	161	250	325
	27848	82	162	277	429
	26902	74	136	226	334
	29551	73	141	241	359
	25380	67	129	223	345
	27031	77	146	244	372
	27943	80	158	270	408
Instron 8800 1000 kN	28426	77	149	256	381
	27866	94	176	292	415
	30646	76	141	237	356
	27860	75	149	266	414
	28186	82	158	271	414
	30030	86	166	290	454
	30102	81	158	278	432
	31719	92	165	257	351
	27448	63	120	206	317
MTS 243 100 kN	27577	86	163	275	415
	29808	84	165	287	443
	26612	43	97	195	306
	27972	52	115	224	349
	28142	69	157	298	457
	29655	74	166	310	474
	28070	55	124	236	363
	29110	59	137	265	409
	29687	64	138	254	384
MTS 244 250 kN	28420	51	109	203	318
	30475	55	117	225	333
	25017	48	101	193	304
	27240	67	137	245	388
	28661	77	164	296	452
	26143	63	119	198	288
	27246	71	142	249	389
	26488	67	135	233	351
	29283	75	153	274	420
	27383	59	120	209	324
	26855	66	133	233	370
	28590	68	130	232	367
	27163	60	123	219	343

Influence of the number of replicates in a batch on apparent variability in FRC and FRS performance assessed using ASTM C1550 panels

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ABSTRACT: When the mechanical performance of Fibre Reinforced Shotcrete (FRS) and Fibre Reinforced Concrete (FRC) is required to satisfy minimum limits, it is normal practice to conduct performance tests on the materials as sprayed or cast either as part of pre-qualification trials or Quality Control (QC) programmes. The ASTM C1550 round panel test is commonly used as the means of assessing the performance of FRS and FRC with regard to energy absorption (toughness). It is widely recognized that all test results exhibit inherent variability. Therefore, several specimens are required to be produced for a given batch of FRS or FRC and tested at the same age to obtain a reasonably reliable estimate of mean performance. However, it is not widely recognized that the common method of calculating the sample standard deviation for a set of replicate test results is a *biased estimate* of the true within-batch variability. Moreover, the true standard deviation for a population of specimens is always greater than the standard deviation calculated for a small number of specimens. The current study examines how the calculated sample variability (expressed as coefficient of variation) changes with the number of specimens tested within a batch of nominally identical FRS and FRC to determine whether a systematic relation exists between the number of specimens and the estimator of the sample variability. It also examines whether a generic correction can be applied to the estimate of the sample coefficient of variation calculated for a relatively small number of specimens to obtain a more reliable estimate of the true variability of a population of ASTM C1550 test results.

1 INTRODUCTION

The ASTM C1550 round panel test (ASTM, 2008) is now well established as a means of assessing post-crack performance for fibre reinforced shotcrete (FRS) and fibre reinforced concrete (FRC), either as part of pre-qualification trials or quality control (QC) programmes, especially in Australia. While the within-batch variability (commonly measured as the coefficient of variation, or sample standard deviation divided by the sample mean) of results obtained by this test method is relatively low, there remains an inherent level of variability that necessitates the use of at least two specimens to represent the performance of a batch of FRC or FRS at a given age. The coefficient of variation (COV) in energy absorption at 40 mm central deflection obtained for pairs of specimens is estimated to lie between 8% (Bernard, 2002) and 11% (Bernard et al, 2010) depending on how it is assessed.

The process of FRC or FRS specimen production is implicitly taken to be a stochastic process (that is, a process subject to variations in outcomes)

resulting in specimens that represent independent (that is, uncorrelated) observations of a performance parameter of interest. When ASTM C1550 panels are used to assess performance, the parameter of interest is most commonly energy absorption at 40 mm central deflection. The observations of performance, denoted

$$X_1, X_2, \dots, X_n \quad (1)$$

are taken to represent n test results extracted without replacement from a population of all possible results. In engineering practice it is common to seek information about the mean level of performance and the variation in this level of performance in order to construct a confidence interval on the mean level of performance for a batch of FRS or FRC. However, to calculate the mean and variation in performance for a set of specimens (observations) representing a stochastic process, it may be necessary to make some assumptions about the process that may not strictly be true but which allow determination of estimates of the mean

and variability (Law & Kelton 2000). One such common assumption regarding test results is that they are *covariance-stationary*, meaning that the mean and variance do not depend on where samples are taken from within a batch. This is equivalent to assuming that the mean and variance for a sample should be the same as the mean and variance for the entire underlying population.

If $X_1, X_2, \dots, X_n$ are independent random observations with a finite population mean μ and variance σ^2 (which remain unknown), we commonly seek estimates of μ and σ^2 in the form of a sample mean,

$$\bar{X}(n) = \frac{\sum_{i=1}^n X_i}{n} \quad (2)$$

and a sample variance

$$S^2(n) = \frac{\sum_{i=1}^n [X_i - \bar{X}(n)]^2}{n-1} \quad (3)$$

both of which are taken to be “unbiased” estimators for independent observations. An estimator of a parameter is unbiased if the mean of the sampling distribution for the estimator is equal to the population value of the parameter. If the assumption that $X_1, X_2, \dots, X_n$ are independent random observations is not true, the sample mean is still an unbiased estimator of μ but the sample variance $S^2(n)$ is no longer an unbiased estimator of σ^2 .

Another factor that is seldom appreciated is that the sample standard deviation $S(n)$ is not an unbiased estimator of σ . This is due to the nonlinearity of the square root operator, a problem that will be discussed later in this paper. Finally, the estimate of $S(n)$ depends on the number of replicate tests even for data conforming exactly to the normal distribution (Anderson, 1994). At present, it is unclear whether or not data produced using the ASTM C1550 method conform to the normal distribution.

For the reasons described above, the following study was developed with two objectives in mind. The first was to examine the probability density functions for the performance parameters measured by ASTM C1550 and determine which of several commonly used functions best describes the distribution of observed data within a batch. The second was to examine whether a systematic relationship exists between within-test variability of ASTM C1550 performance parameters and the number of replicate test results used to calculate variability.

2 EXPERIMENTAL PROGRAMME

It is commonly assumed that within-batch data obtained for sets of concrete specimens are dis-

tributed normally about the mean, that is, conform to the Gaussian distribution. The applicability of many aspects of the standard approach to statistical assessment of experimental data is predicated on the assumption that the test data conform to the Gaussian distribution. The assumption of normality has not been adequately examined for results obtained using the ASTM C1550 panel test. To address this deficiency, five sets of 50 round panels were produced and tested in sequence to examine the characteristic distribution of results. The FRS and FRC mixtures were selected to represent a broad range of mixtures that might conceivably be tested using ASTM C1550, but they do not represent an exhaustive list of possible mixtures. The study was limited to five mixtures and 50 replicates within each set due to resource limits.

These data were also used to test whether the estimate of the mean and standard deviation obtained for cracking load and post-crack energy absorption data varied with the number of specimens.

2.1 Production of specimens

Five sets of round panels were cast or sprayed using both FRC and FRS mixtures, as summarised in Table 1. Each casting session involved delivery of a 3 m³ load of concrete in a 5.6 m³ agitator to which the fibres were added on top of the concrete. The load was then mixed vigorously for 15 minutes before being discharged via the chute into forms that were lying directly on the ground. Specimens were vibrated using an internal 38 mm poker vibrator, screeded, floated, and left to harden under plastic sheeting. The fifth set (PDP5) was manually sprayed using a set accelerated mixture (Figure 1). The casting or spraying sequence of each specimen was carefully noted and all specimens were individually identified. All specimens were cured together in lime-saturated water for 3 months prior to testing. Panel specimens were tested in random order and were in a surface moist condition at time of test. A 100 kN MTS 244.21 testing machine was used to test all the specimens (Figure 2).

2.2 Probability distribution results

Each ASTM C1550 panel test result comprised a load-deflection curve up to 40 mm central

Table 1. FRC and FRS mixtures used in this study.

Set	Fibre	Mix details
PDP1	40 kg/m ³ Novotex 1050	32 MPa cast FRC
PDP2	40 kg/m ³ Dramix RC 65/60	32 MPa cast FRC
PDP3	4 kg/m ³ Barchip Macro	32 MPa cast FRS
PDP4	7 kg/m ³ Barchip Shogun	50 MPa cast FRS
PDP5	6 kg/m ³ Barchip Shogun	50 MPa FRS



Figure 1. Spraying panel specimens.



Figure 2. ASTM C1550 round panel test.

deflection. The performance of each panel was summarized as the peak load sustained upon cracking of the concrete matrix, and energy absorption at 5, 10, 20, and 40 mm central deflection. A small number of specimens suffered a single diametral crack rather than the required three radial cracks; these invalid results were discounted from the summarized record.

The results were analysed using the method of maximum likelihood estimation (MLE) to determine whether the results for peak load and energy absorption parameters were distributed according to the Gaussian, lognormal, or Weibull probability density functions (Kendall & Stuart, 1979; Law & Kelton, 2000). Table 2, for example, shows the results for the energy absorption at 40 mm of deflection. This shows that the Gaussian function was the best fit distribution for three of the five data sets as indicated by a more positive value of the likelihood indicator (the best result is shown in bold). However, the other two functions exhibited similar goodness-of-fit. Figure 3 shows the best fit Gaussian cumulative probability densities (CPD) for the 40 mm energy absorption data. It is seen that the

Table 2. Likelihood estimators for Gaussian, lognormal, and Weibull probability density functions for energy absorption at 40 mm deflection from all sets.

Parameter	Likelihood (MLE)		
	Gaussian	Lognormal	Weibull
PDP 1			
Energy at 40 mm	-275.35	-276.23	-275.94
PDP 2			
Energy at 40 mm	-292.29	-292.96	-292.94
PDP 3			
Energy at 40 mm	-218.16	-217.64	-221.52
PDP 4			
Energy at 40 mm	-272.10	-272.34	-275.09
PDP 5			
Energy at 40 mm	-258.00	-261.34	-257.03

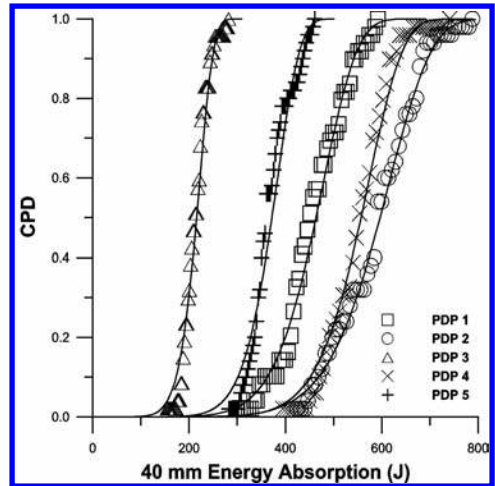


Figure 3. Cumulative probability density (CPD) for energy absorption at 40 mm for all PDP data sets.

fit is not as good for series PDP 3 and PDP 5 as for the other three series, which is consistent with Table 2. Similar results were obtained for the other parameters.

This analysis did not reject the hypothesis that the Gaussian (or normal) distribution is an adequate representation of the probability density function for peak load and energy absorption parameters obtained using ASTM C1550 panels. However, some of the sets of results conformed equally well to more than one of the probability functions examined. In summary, while the analyses did not prove conclusively that the normal distribution is the best-fit function in all cases, there was evidence that it is a reasonable representation of the distribution of results. Thus for the remainder of the analyses, it was assumed that the data were normally distributed.

3 WITHIN-BATCH VARIABILITY AND SAMPLE SIZE

The second stage of analysis of the ASTM C1550 panel results involved an assessment of the influence of the number of replicate test results on the computed within-batch variability. When ASTM C1550 panels are produced as part of a QC programme the number of specimens seldom exceeds three. This represents only a small portion of the total load supplied. Because cracking load is determined by the strength of the concrete, which in turn is influenced by the adequacy of mixing, and post-crack performance is directly influenced by fibre dosage and concrete strength (among other things), it is important to know whether the small sample of FRS contained in these test specimens adequately represents the whole load. One way to check this is to determine how variability changes with the number of samples taken from a given load and the sequence in which the samples were taken.

Two methods of analysing the data were used to assess the influence of the number of replicates on the apparent variability. In the first method, the casting order of each specimen was noted, and the mean and standard deviation were determined using the results from *successive* specimens extracted from the total number tested, which was about 50 for each set. Hence, the mean and standard deviation were found for every possible successive pair ($n = 2$) of panels from panels 1 and 2 to 49 and 50, and the averages of all the means and of all the standard deviations for these pairs were calculated. The same was done for every possible successive set of three ($n = 3$) panels, then four panels, and so on up to the last analysis, which included all the panels within each set.

In the second method of analysis, all possible groups of two specimens, three specimens, etc., up to 49 or 50 specimens, were drawn *at random* from each data set and used to calculate the mean and average within-batch COV. This method of analysis would have discounted any systematic influence of mixture variation as the concrete was discharged from the agitator and thus differences between successive and random sampling could possibly indicate poor mixing.

3.1 Successive sampling results

The results for the first method of analysis involving successive sampling of data are presented in Figures 4 through 8. The mean for all successive groups of panels within each set did not change with the specimen number n , but the within-test variability (expressed as COV in the figures) increased substantially as n increased from 2 to 10. For $n > 10$, the COV fluctuated but did not vary in a consistent manner. This behaviour occurred

for the cracking load as well as for the energy absorption parameters, suggesting that the systematic increase in COV between $n = 2$ and $n = 10$ was not related to fibre distribution. Although fibre count on each crack face is assessed manually and is known to be corrupted by the incidence of fibre rupture, the variation in COV for this parameter was also examined (Figure 9) and was found to increase with n , especially for $n < 10$.

3.2 Random sampling results

When the performance data were analysed using the second method (in which specimens were drawn at random from the total pool of available results), the within-batch COV was found to increase monotonically with n (see Figures 10 to 14). Figure 15 compares the average COV in energy absorbed at 40 mm deflection for the five sets of panels when specimens were selected randomly versus successively. This shows that selection of random specimens results in a slightly higher COV for small n , but a slightly lower COV for large n . This suggests that some ‘drift’ occurred in the values of the performance parameters for specimens made as concrete was discharged from the agitator. Thus, the performance of a small number of specimens made from one portion of a batch (as is normally done) may depend on where in the batch the sample was taken. This ‘drift’ occurred despite 15 minutes of rigorous agitation.

Figure 15 also reveals that the number of specimens n has a much more significant influence on the calculated COV than does the method of sampling (random versus successively chosen specimens). This is also apparent in the results presented in Tables 3 and 4, which show the average COVs for the different parameters for n equal to 2, 10 and ~ 50 .

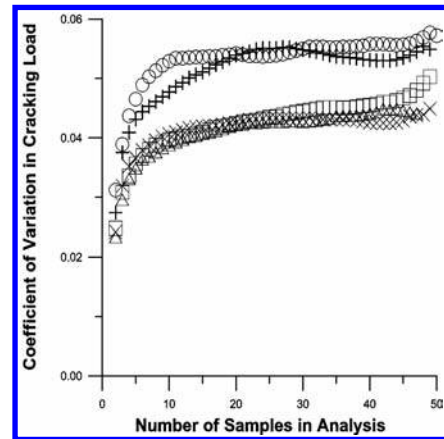


Figure 4. Coefficient of variation in cracking load as a function of number of specimens, n , successively sampled and analysed in each set. Symbols are the same as in Figure 3.

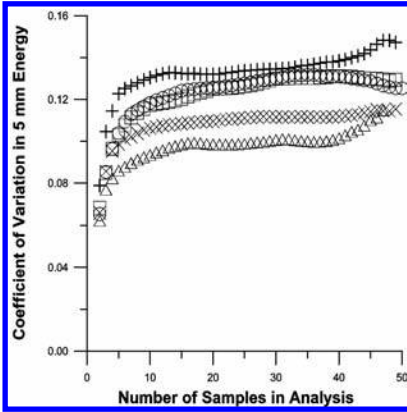


Figure 5. Coefficient of variation in energy absorption at 5 mm deflection as a function of the number, n , of specimens successively sampled and analysed in each set.

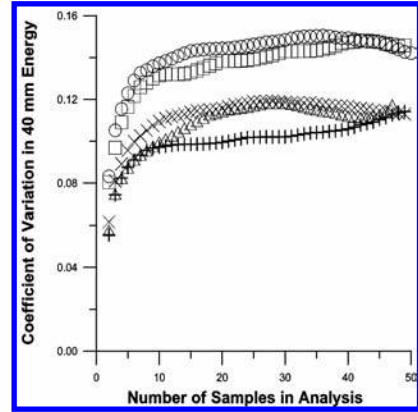


Figure 8. Coefficient of variation in energy absorption at 40 mm deflection as a function of the number, n , of specimens successively sampled and analysed in each set.

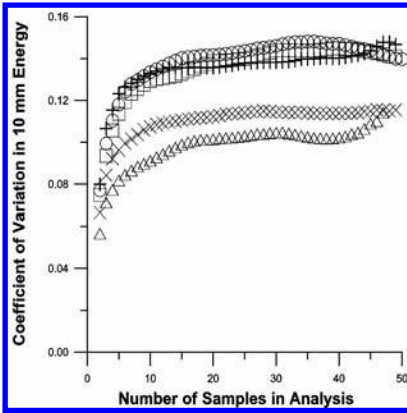


Figure 6. Coefficient of variation in energy absorption at 10 mm deflection as a function of the number, n , of specimens successively sampled and analysed in each set.

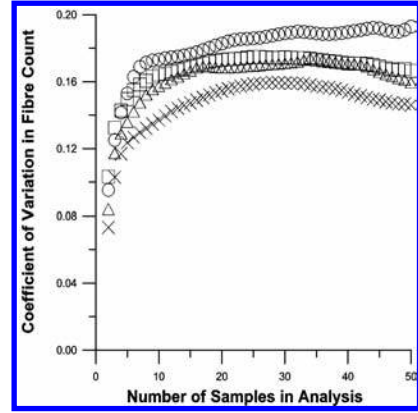


Figure 9. Coefficient of Variation in fibre count as a function of the number, n , of specimens successively sampled and analysed in each set.

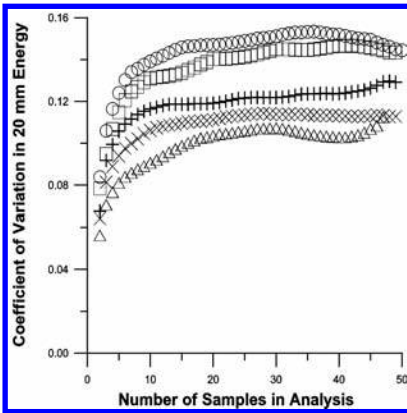


Figure 7. Coefficient of variation in energy absorption at 20 mm deflection as a function of the number, n , of specimens successively sampled and analysed in each set.

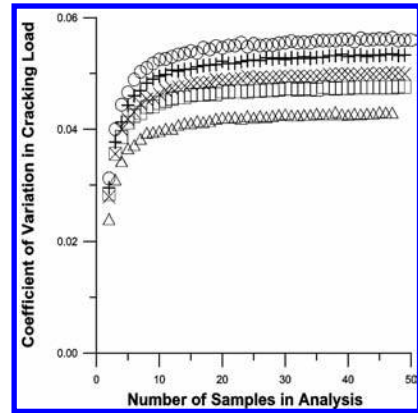


Figure 10. Coefficient of variation in cracking load as a function of number of specimens, n , randomly selected and analysed in each set.

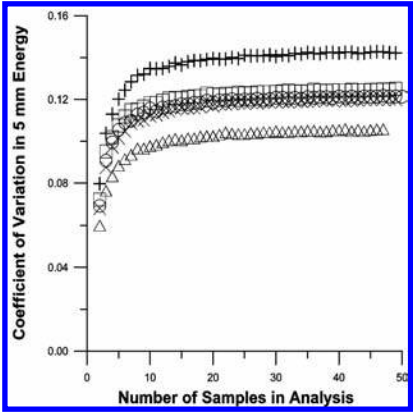


Figure 11. Coefficient of variation in energy absorption at 5 mm deflection as a function of number of specimens, n , randomly selected and analysed in each set.

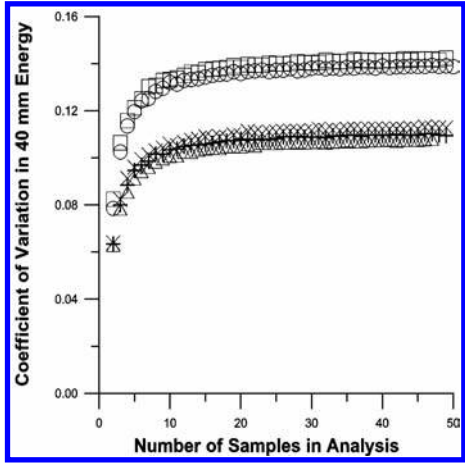


Figure 14. Coefficient of variation in energy absorption at 40 mm deflection as a function of number of specimens, n , randomly selected and analysed in each set.

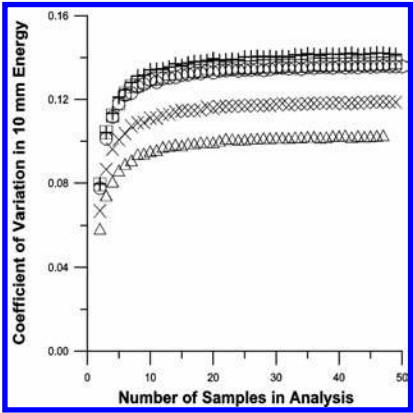


Figure 12. Coefficient of variation in energy absorption at 10 mm deflection as a function of number of specimens, n , randomly selected and analysed in each set.

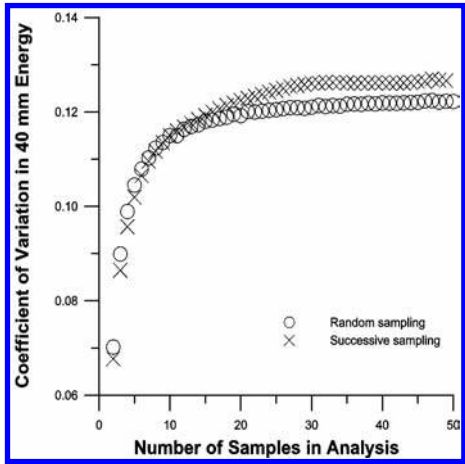


Figure 15. Comparison of the average coefficient of variation in energy absorption at 40 mm deflection as a function of number of specimens, n , for all five mixtures for both randomly and successively selected specimens.

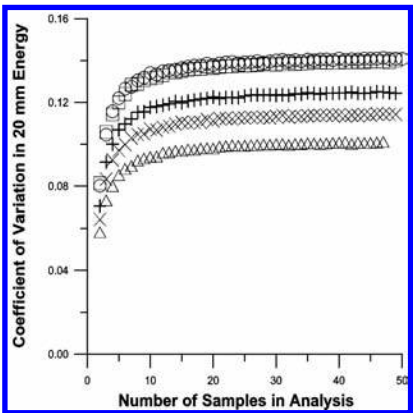


Figure 13. Coefficient of variation in energy absorption at 20 mm deflection as a function of number of specimens, n , randomly selected and analysed in each set.

Table 3. Average COVs (in percent) at selected specimen numbers n for all five mixtures when assessed using successive sampling. V_2/V_{50} is the COV for $n=2$ divided by COV for $n=50$.

n	Crack load	Energy absorption				Fibre count
		5 mm	10 mm	20 mm	40 mm	
2	2.6	6.8	7.1	7.0	6.8	7.9
10	4.4	11.3	11.9	11.7	11.5	14.3
~50	5.0	12.6	13.1	12.9	12.7	15.2
V_2/V_{50}	0.52	0.54	0.54	0.54	0.54	0.52

Table 4. Average COVs (in percent) at selected specimen numbers n for all five mixtures when assessed using random sampling.

n	Crack load	Energy absorption				Fibre count
		5 mm	10 mm	20 mm	40 mm	
2	2.8	7.0	7.2	7.1	7.0	8.6
10	4.6	11.6	12.0	11.7	11.5	14.0
~50	5.0	12.3	12.7	12.4	12.2	15.0
V_2/V_{50}	0.56	0.57	0.57	0.57	0.57	0.57

The last row of each table shows the ratio of the COV for $n=2$ to the COV for $n=50$. The significance of this ratio is discussed in the next section.

4 DISCUSSION

The outcome of this investigation indicated that the calculated coefficient of variation in test results increased with the number of specimens, especially for $n < 10$, regardless of which parameter was used as the basis of performance. The increase also occurred when replicate test results were obtained by either successive or random sampling. This section discusses this behaviour in light of the known characteristics of a normal (Gaussian) probability distribution.

4.1 Jensens' inequality

The sample variance calculated by Eqn. (3) is an unbiased estimate of the population variance σ^2 . This means that the expected value of S_n^2 should be equal to σ^2 (which is expressed as $E[S_n^2] = \sigma^2$ where " $E[\]$ " means the expected or mean value). Typically, textbooks on elementary statistics mention that S_n is not an unbiased estimator of σ , but do not explain why this is so (e.g., Miller & Freund, 1965). The explanation has to do with the fact that non-linear functions such as the square root are not commutative with respect to taking the expectation (Wikipedia, 2009). This characteristic is expressed formally by Jensen's inequality, which states that if $f(X)$ is a continuous concave function (second derivative is negative), then the following inequality holds (Reid, 2009):

$$E[f(X)] \leq f(E[X]) \quad (4)$$

To illustrate the meaning of Eqn (4), consider two functions: $f_1(X) = 6X$ and $f_2(X) = \sqrt{X}$. The first function is a linear function and the second one is a concave function. Table 5 lists several values of X and the expected values (means) of X and of the two functions. The last line shows the value when the function is applied to the mean

Table 5. Example to illustrate Jensen's inequality for a non-linear concave function.

Parameter	X	$f_1(X) = 6X$	$f_2(X) = \sqrt{X}$
	2	12	1.414
	4	24	2.000
	6	36	2.449
	8	48	2.828
	10	60	3.162
	12	72	3.464
	14	84	3.742
	16	96	4.000
	18	108	4.243
$E[X]$	10		
$E[f_1(X)]$		60	3.034
$f_1(E[X])$		60	3.162

value of X . It is seen that for the linear function, the mean of the function is equal to the function of the mean. On the other hand, the mean of $\sqrt{X}$ (3.034) is less than the square root of the mean value of X (3.162). This illustrates Jensen's inequality for a concave function.

If we apply Eqn. (4) to the case of calculating the sample standard deviation from the sample variance, the function $f(X)$ is the square root function, and we obtain:

$$E\left[\sqrt{S^2(n)}\right] \leq \sqrt{E[S^2(n)]} \quad (5)$$

The term on the right side is equal to $\sqrt{\sigma^2}$, and thus we have the following:

$$E[S(n)] \leq \sigma \quad (6)$$

Equation (6) states that the expected (or average) value of the sample standard deviation is less than the population standard deviation.

4.2 Correction of calculated standard deviation

Based on Eqn. (6), the expected value of the standard deviation computed for n replicates underestimates the population standard deviation. The degree of under-estimation decreases as the sample size increases. We can therefore say that $S_n = c_4\sigma$, where c_4 is a factor less than one, which approaches unity with increasing n . An unbiased estimate of the population standard deviation would be obtained by dividing the calculated sample standard deviation by c_4 ; that is, $\sigma = S_n/c_4$. This same correction factor would apply to the sample COV because the sample mean does not change with n .

For a normal distribution, the correction factor c_4 has been determined to equal a ratio of Gamma functions subject to n (Holtzman 1950; Cureton 1968). This correction factor, however, is approximated

closely (maximum error is 0.3% at $n=3$) by the following simple hyperbolic equation (ASTM 2009):

$$c_4 = \frac{n-1}{n-0.75} \quad \text{for } n=2 \quad (7)$$

Figure 16 is a plot of the correction factor c_4 given by Eqn. (7) as a function of sample size.

For $n=2$, the value of c_4 is equal to 0.8, while for $n=6$ it has a value of 0.95. It is emphasized that the correction factor in Figure 16 is only applicable if the underlying population is normally distributed.

4.3 Analysis of results from this study

As can be seen in Tables 3 and 4, the ratio of COV for $n=2$ to the COV for $n=50$ is consistently less than the expected value of 0.8 that is applicable for a normal distribution. Thus the following function was chosen for regression analysis to determine whether it could describe the variation of COV with n for the various performance parameters:

$$\text{COV} = \text{COV}_{\text{inf}} \frac{n-a}{n-b} \quad (8)$$

The term COV_{inf} is the limiting value of the COV for large samples sizes, which is determined from the least-square regression. Figure 17 shows the fit of Eqn. (8) to the COV values for the different performance parameters for mixture PDP1. It is seen that the fit is remarkably good. A similar good fit was obtained for the other mixtures.

Table 6 shows the resulting best-fit values of the terms in Eqn. (8) for all the mixtures and performance parameters. The last column shows the residual standard deviation (RSD) of the fit, which represents the average deviation of the points from the best-fit curve. The small values of RSD show that Eqn. (8) is a good representation of the relationship between the sample COV and the number of replicate test results.

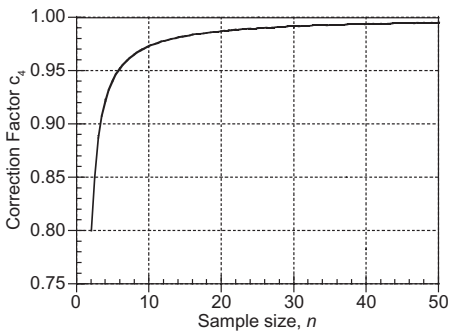


Figure 16. Value of correction factor c_4 according to Eqn. (7).

Figure 18 shows the corresponding paired values of a and b listed in Table 6. It is apparent that there is a linear relationship between these two coefficients: a high value of a is associated with a high value of b . While it appears that there are widely different values for these two coefficients, it can be shown that they result in similar hyperbolic curves. The only difference is that higher values of a and b result in slightly steeper slopes for small values of n . A surprising result is that the pairs of coefficients in Table 6 result in similar values of the ratio of COV for $n=2$ to V_{inf} , which is consistent

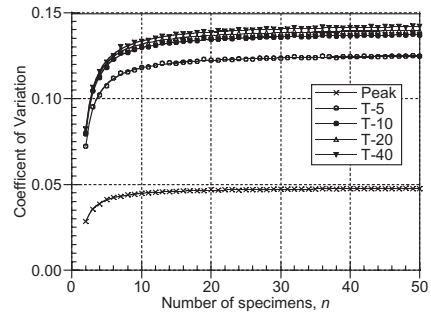


Figure 17. Fit of Eqn. (8) to the COV values for mixture PDP1.

Table 6. Best-fit values of terms in Eqn. (9) and the residual standard deviation of the fitted curve.

Mixture	Parameter	COV_{inf}	a	b	RSD
PDP1	Crack	0.048	0.966	0.238	0.00013
	T5	0.126	1.141	0.498	0.00040
	T10	0.139	1.133	0.486	0.00041
	T20	0.142	1.083	0.407	0.00040
	T40	0.144	1.044	0.332	0.00043
PDP2	Crack	0.057	0.965	0.098	0.00021
	T5	0.123	1.155	0.493	0.00038
	T10	0.138	1.160	0.506	0.00038
	T20	0.143	1.154	0.486	0.00037
	T40	0.141	1.134	0.440	0.00038
PDP3	Crack	0.044	0.971	0.132	0.00018
	T5	0.108	0.856	-0.044	0.00036
	T10	0.105	0.864	-0.033	0.00032
	T20	0.103	0.877	0.014	0.00028
	T40	0.110	0.862	0.007	0.00027
PDP4	Crack	0.051	0.948	0.076	0.00016
	T5	0.122	1.000	0.206	0.00033
	T10	0.121	1.024	0.240	0.00035
	T20	0.116	1.027	0.244	0.00036
	T40	0.115	1.015	0.237	0.00037
PDP5	Crack	0.054	0.882	-0.065	0.00018
	T5	0.145	1.024	0.222	0.00053
	T10	0.144	1.066	0.305	0.00047
	T20	0.127	1.055	0.299	0.00037
	T40	0.112	1.002	0.232	0.00035

with V_2/V_{50} shown in Table 4. The values of this ratio are shown in Figure 19 for the different performance parameters and mixtures. It is seen that the ratio varies between 0.54 and 0.59.

Given that the shapes of the best fit curves represented by Eqn. (8) were similar for all cases, the values of the COV for each case shown in Figures 10 to 14 were normalized by dividing by the corresponding values of COV_{inf} given in Table 6. The resulting normalized values are shown in Figure 20. The following equation represents the best fit to the normalized values (the best-fit values shown in Figure 21 have been rounded to two decimal places):

$$\frac{COV}{COV_{inf}} = \frac{n - 1.02}{n - 0.24} \quad (9)$$

By rearranging Eqn. (9), we can estimate the population coefficient of variation based on the coefficient of variation determined from a small sample size.

Figure 21 compares Eqn. (7) with Eqn. (9). The results of this study show that the c_4 correction

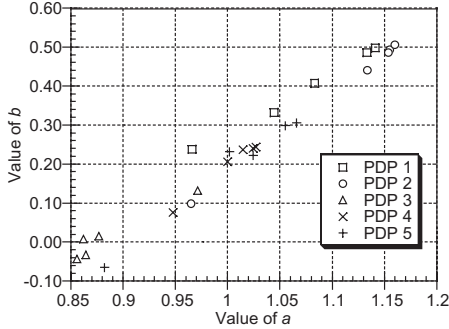


Figure 18. Best fit values of a and b listed in Table 6.

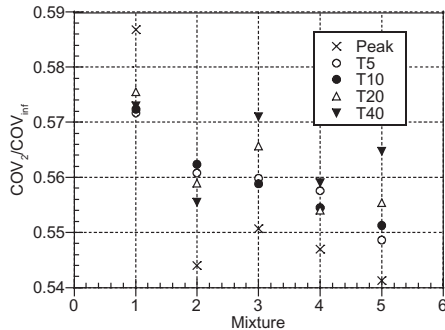


Figure 19. Ratio of COV for $n = 2$ to COV_{inf} based on the best-fit values of a and b .

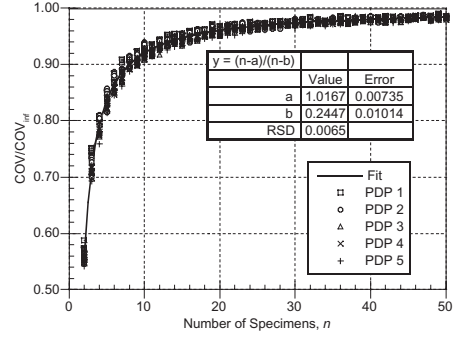


Figure 20. Normalized values of COV and resulting best-fit curve.

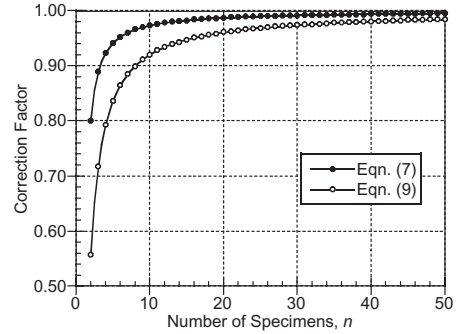


Figure 21. Comparison of the correction factors for sample size based on Eqn. (7) and Eqn. (9).

factor given by Eqn. (7) is not applicable to this data set. For small sample sizes, the within-test variability for the ASTM C1550 test will be under-estimated more than is expected based on the conventional value of c_4 derived for a normal distribution.

At this time, there is no explanation why these data departed greatly from the expectations for a normally distributed population. It is suggested that this may be due to the fact that the probability density distribution for each data set PDP1 to PDP5 deviated from a perfect fit to the normal distribution. Table 2 shows that the Maximum Likelihood Estimator (MLE) for the Gaussian, lognormal, and Weibull function fits were nearly the same for each data set. This indicates that none of the functions were a particularly accurate fit. Additional study is clearly necessary.

4.4 Significance of findings

The within test variability is used typically to construct a confidence interval for the population mean based on the sample mean using the t -statistic. The upper

and lower limits of the confidence interval are given by the following expression (Law & Kelton, 2000):

$$\bar{X}(n) \pm t_{n-1, 1-\alpha/2} \sqrt{\frac{S^2(n)}{n}} \quad (10)$$

In Eqn. (10), α is the significance level, which is typically taken as 0.05. Equation (10) is based on estimating the population variance from the sample variance. The t -statistic is a large number for small sample sizes and this is believed to take into account the dependence of the sample variance on the sample size. The problem is that the data from this study appear to show that the computed within-test variability has greater sample size dependence than is expected for a normal distribution. Again this requires additional study.

It appears that the sample size correction factor c_4 for the sample standard deviation is used in statistical practice only with regard to control chart analysis (ASTM, 2009). When plotting within-test variability on a control chart, the computed sample standard deviation is corrected for sample size for comparison to the target population standard deviation.

5 CONCLUSIONS

One objective of this study was to examine the probability density functions for FRS performance parameters measured using round panels in accordance with ASTM C1550. For this purpose, 50 panels were tested for each of five fibre reinforced concrete mixtures. The analysis indicated no strong evidence to reject a Gaussian distribution, but it was not always the best-fit distribution.

A second objective was to examine how the mean and the variability of the performance parameters varied with the number of replicate test results. For this purpose, different numbers of test results were drawn from the population of 50 tests for each mixture. It was found that the method of sampling (successive or random) did not have a major effect on computed variability. The mean was found to be independent of the number of test results. The coefficient of variation, on the other hand, was found to have a strong and consistent dependence on the number of replicate test results used to compute the COV. This dependence was greater than expected for normally distributed data. The implication is that the usual practice of estimating confidence intervals for the population mean may underestimate the uncertainty. In addition, the

COV estimated from two or three replicates tests will greatly under-estimate the true COV. Additional study is required to gain a better understanding of the significance of these findings.

ACKNOWLEDGEMENT

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REFERENCES

- American Concrete Institute 506.1R, 2008. *Guide to Fiber-Reinforced Shotcrete*, ACI, Farmington Hills.
- Anderson, T.W. 1994. *The Statistical Analysis of Time Series*, John Wiley, New York.
- ASTM International, 2009. "Manual on Presentation of Data and Control Chart Analysis", MNL7A, West Conshohocken, Pa., www.astm.org.
- ASTM International, 2008. Test Method C1550, "Standard Test Method for Flexural Toughness of Fiber Reinforced Concrete (Using Centrally Loaded Round Panel)", ASTM, West Conshohocken, Pa.
- Bernard, E.S. 2002. "Correlations in the behavior of fiber reinforced shotcrete beam and panel specimens", *Materials and Structures*, RILEM, Vol. 35, April, 156–164.
- Bernard, E.S., Xu., G.G., & Carino, N.J. 2010. "Precision of the ASTM C1550 panel test and field variation in measured FRS performance", *Shotcrete: Elements of a System*, Taylor & Francis, London.
- CIA, 2008. *Shotcreting in Australia: Recommended Practice*, Concrete Institute of Australia & AuSS, Sydney.
- Cureton, E.E. 1968, "Unbiased Estimation of the Standard Deviation", *The American Statistician*, Vol. 22, No. 1, 22 p.
- EFNARC, 1996. *European Specification for Sprayed Concrete*, European Federation of National Associations of Specialist Contractors and Material Suppliers for the Construction Industry.
- Holtzmann, W.H., 1950, "The Unbiased Estimate of the Population Variance and Standard Deviation", *American Journal of Psychology*, 64, 615–617.
- Kendall, M. & Stuart, A. 1979. *The Advanced Theory of Statistics, Volume 2: Inference and Relationship*, 4th Ed., Charles Griffin & Co., England.
- Law, A.M. & Kelton, W.D., 2000. *Simulation Modelling and Analysis*, 3rd Ed., McGraw-Hill New York.
- Miller, I. & Freund, J.E., 1965, *Probability and Statistics for Engineers*, Prentice-Hall, Englewood Cliffs, NJ.
- Reid, M. 2009, "Behold Jensen's Inequality", <http://mark.reid.name/iem/behold-jensens-inequality.html>
- Wikipedia, 2009, "Unbiased Estimation of Standard Deviation", http://en.wikipedia.org/wiki/Unbiased_estimation_of_standard_deviation

Round and square panel tests—effect of friction

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ABSTRACT: The tradition in Norway has been to use round 600 mm panels (on continuous simple support of wood) for determination of the energy absorption capacity of fibre reinforced sprayed concrete. The new European standard EN 14488-5:2006 describes 600 × 600 mm square panels (on continuous simple support of steel). A comparative study of round and square panel test methods has been conducted. As results were generated it was suspected that friction played a significant role during these tests. The present study was therefore designed specifically to study the influence of friction between the underside of the concrete panel and wooden support fixture. The results reveal that the apparent energy absorption during the tests is greatly affected by friction between the concrete panel and the support fixture. The implication is that there is a need for an assessment of the friction properties of supporting materials.

1 INTRODUCTION

The present investigation is a methodology study in which the effect of friction on the Energy Absorption Capacity (EAC) in round and square panel tests (with continuous simple support) has been investigated. The full investigation is reported in Bjøntegaard (2009). The primary motive for the present study was observations of friction effects during earlier tests (Myren & Bjøntegaard 2008, Bjøntegaard & Myren 2008, Myren & Bjøntegaard 2009). From the literature it has been reported that a 15–20% friction effect exists for ASTM C-1550 round panels with 3-point determinate support (Bernard 2005), but such documentation appears to be absent for continuous support conditions. Any friction forces between the panel and the support fixture during this type of testing, independent of type of support, will be apparent as energy absorption (internal work) within the Fibre Reinforced Sprayed Concrete (FRSC) panel and consequently the EAC determined from a test will be over-estimated. In addition, variations in friction will also contribute to increased scatter.

The scope of the present work was to investigate the friction effect for round and square FRSC panels on continuous support of wood (birch).

2 DIRECT OBSERVATIONS OF FRICTION

During a previous test series (Bjøntegaard & Myren 2008) the support ring failed in several successive tests, see Figure 1. The failure appeared to be a result of friction forces that were transferred to the support. The wooden support ring in the test series

reported above was obviously of inferior quality (chipboard), but the fact that it failed revealed that friction forces appeared to be substantial. If opening of the cracks that form during a test is hindered then tangential (transversal) friction forces are transferred to the support. At the same time, when the panel is pushed downwards by the central load, the cracked sectors rotate and the under-side of the panel slides outwards, hence there will be radial friction forces as well which push the support outwards. This has probably also contributed to the support failure seen in Figure 1.

In the post-cracking stage of a test all transmission of load between panel and support will take place over the sharp crack edges at the inner side of the support, and the reaction force from the support is thus transferred as distinct point-loads.

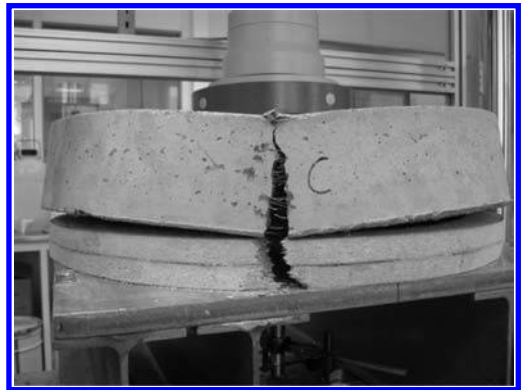


Figure 1. Tensile failure of the support due to friction (Bjøntegaard & Myren 2008).

3 TEST PROGRAM

3.1 Concrete composition, casting and curing

The concrete used in this investigation was ready-mixed and transported by concrete agitator about 30 minutes to a nearby construction area in Oslo, where all casting took place in a tent. The nominal recipe for the concrete mix is shown in Table 1. The mix is a basic sprayed concrete composition, but it was cast and hence no accelerator was added. The nominal water-to-cement ratio ($w/(c + 2s)$) and fibre dosage was 0.42 and 20 kg/m³, respectively.

3.2 Production of panels and specimens

A total of 16 panels were cast, of which 8 were round (Ø600 mm) and 8 were square (600 × 600 mm) panels. Both types of panel had a nominal thickness of 100 mm, and they were cast to their final size. Square and round panels were cast and numbered successively. After casting the panels they were covered with plastic sheeting. De-moulding took place after four days. Fresh concrete air content, density and fibre content were measured. Compressive strength 100 mm cubes were also cast. Slump was not measured, but the workability was found to be very satisfactory. After four days the panels and strength cubes were transported to the Central laboratory of the Norwegian Public Roads Administration where they were stored in water until the day of testing.

3.3 Density and fibre measurements

Density was measured according to EN 12350-6:2000, ie. weighing of FRSC samples in a container with known volume and weight. Two samples, each consisting of 1 litre of wet concrete, were used for density measurements. The weight of the sample was measured. The concrete from the sample was then washed, in portions, over a 1 mm sieve and the fibres were taken out by an electric magnet

and washed completely clean afterwards. When the fibres were completely dry, after a period with air drying (a couple of hours), the total weight of fibres in each sample were determined and the ratio of fibre content (grams) to concrete volume (1 litre) was found. The procedure is in accordance with EN 14488-7:2006.

3.4 Energy absorption capacity measurements

The round panels were tested according to NB7 (Norwegian Concrete Association's publication no.7: 2003) and the square ones according to EN 14488-5:2006. Exceptions from the standards were, however, implemented as follows:

- For the square panels the square support fixture was made of the same wooden material (birch; previously found to be applicable and robust) as for the round panels, and not made of steel (with the bedding material) as described in EN 14488-5. This was done to obtain identical and parallel friction conditions for the two panel types, and thus also to delineate the panel geometry effect for the given supporting material. For both geometries the support fixture had inner/outer dimensions of 500 mm/600 mm.
- Both types of panel were placed with the smooth moulded face towards the support. This is an exception for the round panels since NB7 prescribes the opposite orientation. Again, this was done to ensure as similar test conditions as possible for both panel types.

During testing, the deflection rates for the round and square panels were 1.5 mm/min and 1.0 mm/min, as described in NB 7 and EN 14488-5, respectively. The different deflection rates are believed not to have any impact on the measured energy uptake. The stiffness of the test machine was 235 kN/mm. Two different support conditions were applied: half of the round and half of the square panels were placed directly on the wooden support. This set-up was denoted as the "Standard" condition. For the second set of round and square panels measures were taken to eliminate the friction between the specimen and the support fixture. This set-up is denoted "no friction" conditions, see the following section.

Each of the four sets thus consists of four panels. For three of the four sets the age at testing was 60–61 days. Due to an error in the control and logging system which occurred after the first set of panels were tested, the first set was tested earlier than the other three sets. The test ages therefore became:

- Square panels "standard" condition: 40 days
- Round panels "standard" condition: 60 days
- Square panels "no friction" condition: 61 days
- Round panels, "no friction" condition: 61 days

Table 1. Concrete composition.

Material	Quantity (kg/m ³)
CEM II/A-V 42.5R	226
CEM I 52.5N	225
Silica fume	22
Sand 0–8 mm	1572
Dramix RC 65/35 Steel fibre	20
Superplasticizer (19% weight in solution)	4.1
Retarder (19%)	1.5
Air entraining (1%)	0.9
Free water	208

According to published literature (for instance, DiNoia & Rieder 2004), EAC changes very little between 40 and 60 days (and for the given strength increase). Hence, it is assumed that there was no significant influence.

3.5 Ensuring “no friction” conditions

In the “no friction” tests special measures were taken to eliminate friction, see Figures 2 to 4. Two layers of 1.5 mm strips of polymeric membranes with grease in between were put on top of the support fixture. In order to cope with the outward radial sliding the strips were wider than the width of the support (which is 50 mm wide). The membranes were considered robust and likely to withstand penetration of the sharp edges of the cracks into the support.

About $\frac{3}{4}$ of the width of the upper membrane was cut (from inside and outwards) into “fingers” to eliminate the overall axial rigidity of the membrane. After placing the panels on the support fixture over the two layers of membrane (and grease in between) it was observed that the friction (in the un-cracked state) was very low. The heavy panel could be moved quite easily relative to the support by pushing it sideways with one human finger. It was therefore assumed that there was effectively no friction in these tests. Still, it is likely that a small component of friction was present. It



Figure 2. “No friction” condition. The upper membrane was cut into parallel finger about $\frac{3}{4}$ of the width from the inside and outwards.

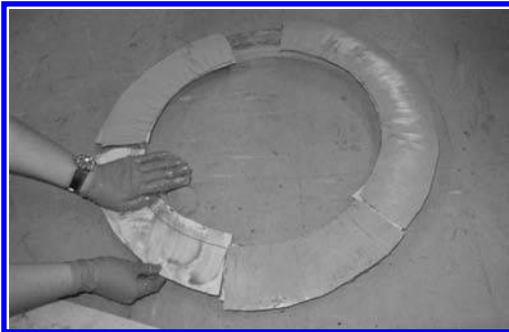


Figure 3. Preparing the round support for the “no friction” condition.

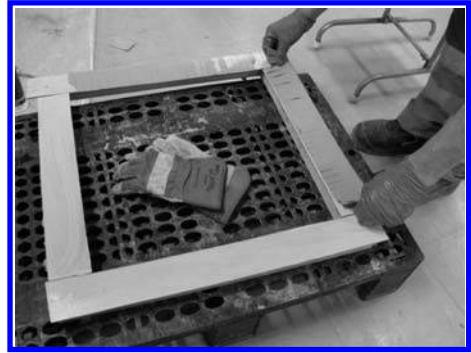


Figure 4. Preparing the square support for the “no friction” condition.



Figure 5. Example, “no friction” condition: rotation of panel, lifting of panel edge, opening of a crack and sliding on the grease layer at the inner side of the support.

is therefore likely that the effect of friction shown in the following is slightly underestimated.

Figure 5 shows opening of a crack where the crack edges rests on the upper membrane “fingers” which again slides on the grease-layer.

3.6 Test procedure

Prior to testing, each panel was taken out of the water bath and transported to the test rig. The test started within 45 minutes, hence the panels were still moist during testing. The procedure was then as follows. The mid-point was marked on the smooth moulded face of the panel. The panel was then placed in the test rig with the smooth moulded face down towards the support fixture, and centred. Two displacement transducers were placed under the centre of the panels. The average of the two transducers forms the signal for control of the load. On the upper side of the panel (the cast side) a load plate (100 mm) was placed at the centre (plus a thin sheet of cardboard). The load plate was round or square depending on the panel type. The load cell was prepared for testing by lowering it to the load

plate until a load of 1 kN is applied to the panel. The test was then started and load and deflection signals were recorded continuously by a computer (1.5 mm/min deflection rate for the round and 1.0 mm/min for the square panels). The test was stopped automatically when the central deflection was 30 mm. The panel was then lifted out of the test rig and the bottom side of the panel was photographed. The panel was then completely broken into pieces along the cracks, and, for each panel, a total of 15–20 thickness measurements were made over the cracks. The thickness was measured with a digital sliding caliper.

The EAC was calculated as the area under the load-deflection curve from zero to 25 mm deflection. The results are corrected for thickness when deviating from 100 mm. For a description of the correction procedure see Thorenfeldt (2006) or Bjøntegaard (2008). The panel thicknesses varied in the range 101 to 105 mm except for one panel being 106.6 mm.

4 RESULTS

4.1 Density, fibre content and comp. strength

Average concrete density and fibre content was measured to be 2280 kg/m³ and 20.8 kg/m³, respectively. This corresponds well with the nominal values. Two 100 × 100 mm cubes were tested after 7 days and two cubes at 28 days concrete age, and the average results were 51 MPa and 72 MPa, respectively.

4.2 Energy absorption capacity (EAC)

The average Coefficient of Variation (COV) for EAC among all four individual sets of panels was 9.7%. For the two individual sets of round panels the average COV was 7.8%, and for the two sets of square panels 11.7%. For the two individual sets at “standard” conditions the average COV was 10.1%, and similarly, for the two “no friction” sets, 9.4%.

Raw-data load-deflection curves from all tests are given in Figure 6 and Figure 7, and average EAC-results for the four sets are given in Table 2. Note that the EAC-results are calculated after correcting for deviating panel thickness as well as for the early non-linear behavior of the load-deflection curve. The latter is likely to come from squeezing of the two membranes (“no friction” condition) as well as some possible compression of the wooden support (both friction conditions). However, the correction procedure showed that the non-linearity, i.e. the error in load train displacement, was less than 1 mm and that the correction procedure had an insignificant impact on the calculated EAC, see Bjøntegaard (2009).

The results show that the average energy absorption in the square and round panels was very simi-

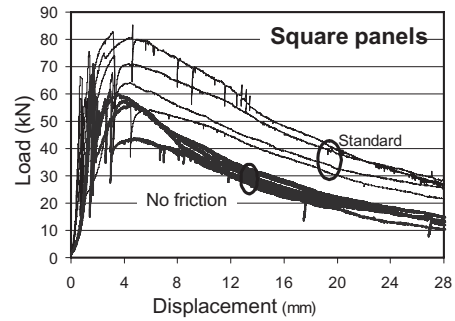


Figure 6. Measured load-deflection curves for all 8 square panel tests.

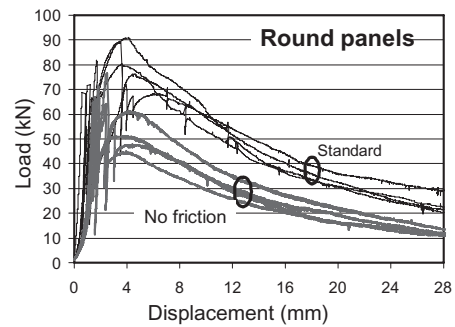


Figure 7. Measured load-deflection curves for all 8 round panel tests.

Table 2. Average EAC-results at 25 mm central deflection.

Panel / friction condition	EAC [J]
Square / standard	1158
Round / standard	1153
Square / no friction	765
Round / no friction	726

lar for similar friction conditions. Furthermore, it is evident that friction had a great impact on the apparent energy absorption. The average EAC between zero and 25 mm central deflection was reduced by 35% when friction was eliminated.

When friction was eliminated in the tests the results also showed, on average, that the maximum load during the test was reduced by 15% and the residual load at 25 mm deflection was reduced by 46%.

5 COEFFICIENT OF FRICTION

5.1 Theory

When calculating the energy absorption in panel tests it is assumed that the external work from the

load P (W_p) equals the internal work experienced by the panel (W_i), hence:

$$W_p = W_i \quad (1)$$

In this way the absorbed energy in the panel (W_i) can simply be calculated as the area under the load-deflection curve (W_p). However, when friction is present it will also contribute work (W_f) and thus the total energy balance is as follows:

$$W_p = W_i + W_f \quad (2)$$

Let us denote the apparent energy absorption and evaluation procedure at standard conditions (for the panel is placed directly on the support) as EAC_{standard} . So, when assuming that EAC_{standard} equals W_p (Equation 1) the EAC_{standard} really consists of two components (Equation 2); the actual internal work within the panel (W_i) and the work due to friction (W_f). It then follows that the actual internal work within the panel (W_i) is:

$$W_i = W_p - W_f = EAC_{\text{standard}} - W_f \quad (3)$$

When it comes to the friction component, the friction force (F) is given by the coefficient of friction (μ) and the normal force (P) according to:

$$F = \mu P \quad (4)$$

The contribution from friction (W_f) in the energy balance will then be the integral of the friction force (F) multiplied with the movement of the panel (w_f) along the contact zone with the support. In our case w_f will consist of a tangential (w_T) and a radial (w_R) movement. As the panel is pushed down by the central force and rotates, the crack edges that form will slide tangentially (due to the opening of the crack) as well as radially (the underside of the panel is pushed outwards) relative to the support, see Figures 8 to 10. The total relative movement dw_f of the panel in the contact zone with the support then becomes:

$$w_F = \sqrt{w_T^2 + w_R^2} \quad (5)$$

A simplified geometrical consideration (Figure 9) gives the following expressions for w_T and w_R , respectively:

$$w_T = \frac{\Delta \cdot h}{L'} \quad (6)$$

where Δ is the central deflection, and

$$w_R = \frac{h}{L'} \left(1 - \frac{\Delta}{h} \right) d\Delta \quad (7)$$

It is assumed that $\sin\alpha = \tan\alpha = \alpha$ for small angles, that four cracks develop and that they stand perpendicular to the support (both for round and square panels). It is also assumed that the cracks open over almost the whole height

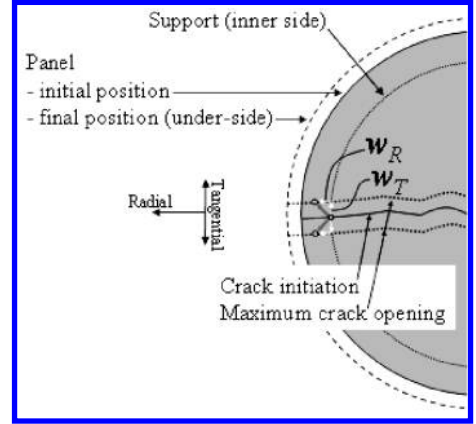


Figure 8. Sketch of panel movements in a Round panel test (seen from the under-side). As a crack opens, the panel slides in radial (w_R) and tangential (w_T) direction relative to the support.

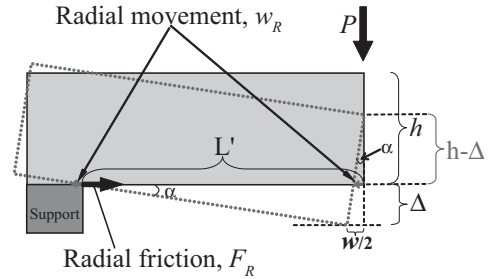


Figure 9. Cross-section of half the panel: illustration of radial movement w_r and radial friction F_r during central loading P and displacement Δ .

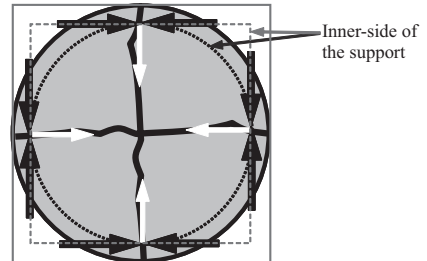


Figure 10. Potential tangential- (black arrows) and radial (white arrows) friction forces in round and square panels with continuous support. Assumption: four perpendicular cracks meeting the support with an angle of 90° .

of the panel except at the very top. During the deflection span from zero to 25 mm deflection it then follows that maximum w_T is 10.0 mm while w_R is 8.7 mm, hence the total movement of the crack edges at the end of the test $w_{F, \max}$ is 13.3 mm (Equation 5).

For an incremental movement dw_F caused by a friction force F the energy from friction W_F can be expressed as:

$$W_F = \int F dw_F = \int \mu P dw_F \quad (8)$$

Considering the above discussion and Equation 3 we then get the following expression for the energy balance:

$$W_i = W_p - W_F = \int P d\Delta - \int \mu P dw_F \quad (9)$$

5.2 Deducing the coefficient of friction

The coefficient of friction between concrete and the wooden support was deduced from the test results. It was assumed that the energy results obtained for the “no friction” condition really had no friction and thus that it represents the internal work W_i within the panel (due to fibre action). The results for the “standard” test condition equal W_p , and Equation 9 then becomes:

$$EAC_{\text{no_friction}} = EAC_{\text{standard}} - W_F \quad (10a)$$

which then gives, in numerical form:

$$EAC_{\text{no_friction}}(\Delta) = EAC_{\text{standard}}(\Delta) - \sum_{\Delta=0}^{25\text{mm}} \mu P dw_F \quad (10b)$$

The only unknown in Equation 10b is the coefficient of friction μ . Before deducing μ the load-deflection record was adjusted for early non-linear behavior; such behavior was most pronounced for the “no friction” tests due to the elasticity of the two layers of membrane that were used on top of the support. Figure 11 shows average accumulated energy curves for all four sets. Again we note that the absorbed energy is rather similar for round and square panels at similar test conditions.

The overall friction effect over the whole deflection span can now be evaluated. The ratio between the energy absorption for “no friction” and “standard” conditions versus central deflection is shown in Figure 9. The ratio is calculated with the use of the average result from each of the two test conditions.

The effect of friction and fibre action is indicated in Figure 12. A constant ratio of 1.0 would

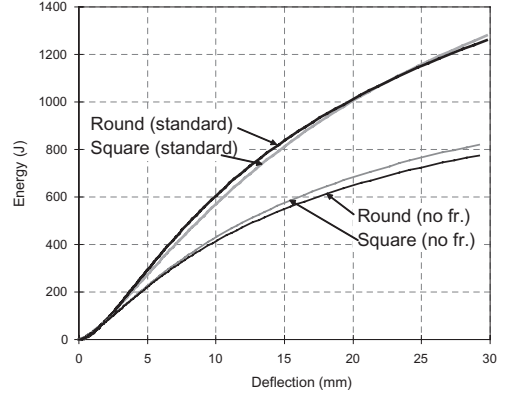


Figure 11. Average accumulated energy absorption for the four sets after adjusting for the early non-linear behavior.

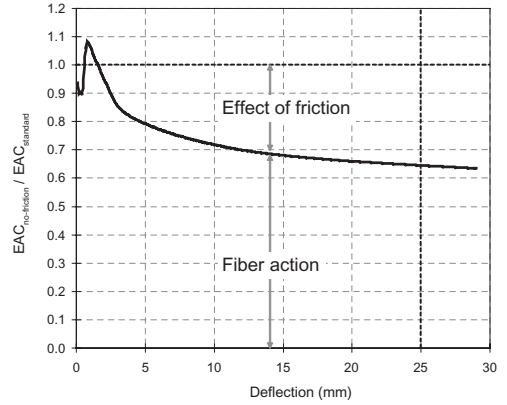


Figure 12. Ratio between average energy absorption (EAC) for the two sets with “no friction” conditions and the two sets with “standard” conditions.

indicate that there was no friction effect in the “standard” set-up, but this is indeed not the case. It is clear that the effect of friction increases with increasing deflection and that the overall friction effect at 25 mm deflection is 35%, which is the same as was shown in Table 2. The implication is that the coefficient of friction increases during the test. Increased deflection means larger and more rotated crack openings which maybe gradually lead to more distinct point-contacts between the panel and the support. It is likely that this leads to some penetration of the crack edges into the softer wooden support; with increased coefficient of friction as a result.

At low deflection levels in Figure 12 (in the pre-cracking phase) the ratio exhibits an irregular behaviour. This is probably mainly due to the sensitivity caused by dividing small numbers by each

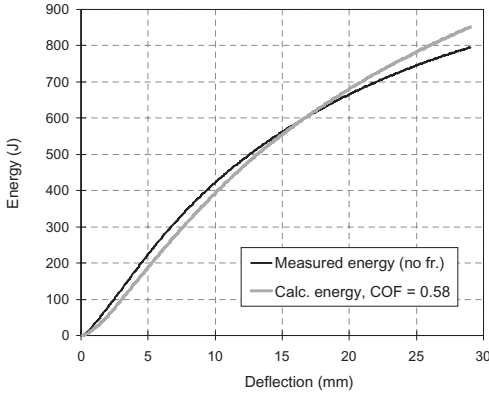


Figure 13. Best fit between calculated- and measured inner energy from the “no friction” tests using a constant coefficient of friction (COF).

other (but probably also partly due to the early adjustments of the load-deflection curves). The different crack propagation behaviour for the two support conditions will also contribute. As more energy is accumulated the curve becomes more “robust”.

The friction coefficient μ was deduced from the test results using Equation 10b and the least square root principle. In the first iteration μ was set to be a constant value. The best fit between the measured internal work ($EAC_{no_friction}$) and calculated internal work ($EAC_{standard} - W_F$) was obtained for $\mu = 0.58$, see Figure 13. It can be seen that this constant μ gives a rather good correspondence. For friction between wood and steel (clean and dry surfaces) the value $\mu = 0.62$ for static friction is given by Kurtus 2005 (no kinetic coefficient is given); this is very close to the constant (and average) value found here. The interaction between static and kinetic friction during a panel test is unclear, but rapid drops in the load at rather high deflection levels in the load-displacement records may indicate that there is an alternation between the two friction types.

In a second iteration μ was expressed by a linear model $\mu(\Delta) = a\Delta + b$. Best fit was obtained for the parameters $a = 0.031$ and $b = 0.33$, see Figures 14 and 15. It can be seen that the correspondence improved somewhat in terms of agreement with the measured curve, implying that μ increases with the deflection level—and μ appears to be very high at high deflection levels. The friction condition in the pre-cracking period at low displacements is somewhat uncertain and it differs from that evident in the post-cracking period for which the calculations are most relevant. Thus, the calculation of μ from zero up to a few millimetres deflection is uncertain; this period is indicated with the grey area in Figure 15.

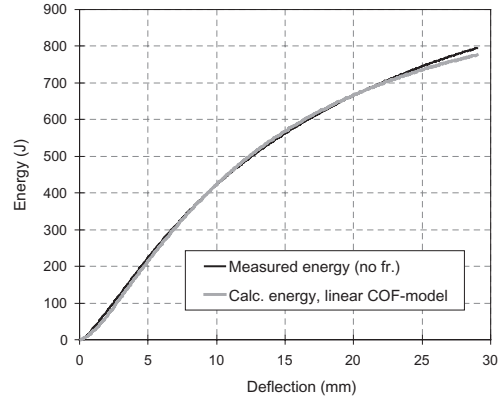


Figure 14. Best fit between calculated- and measured inner energy. The calculation is based on a linear model for the coefficient of friction (COF).

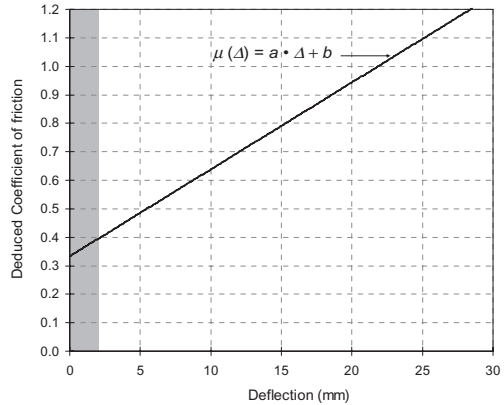


Figure 15. Development of coefficient of friction that gave the best fit for the linear model indicated ($a = 0.031$ and $b = 0.33$). Grey area indicates the pre-cracking period.

5.3 Recent tests with steel supports

The effect of friction in panel tests with a continuous steel support was not investigated here, and, to the author’s knowledge, has not been investigated elsewhere either. Coefficient of friction values found in the literature (Bernard 2005, Cook 1989, Funk 1989, Kurtus 2005) indicate that there is no reason to believe that the effect of friction for steel supports should be any less than shown here for a wooden support, on the contrary. Using a bedding material on top of the steel support fixture (as described in EN 14488-5) probably has an effect, but to what extent is uncertain. It is also somewhat uncertain to which extent bedding material(s) are used in practice. In very recent tests the friction effect in panel tests with continuous steel support has been investigated in the NPRA laboratory. The

results are not evaluated yet, but the findings will be reported in the near future.

6 CONCLUSIONS

The results from panel tests are considered to reflect the load-bearing capacity of fibre reinforced concrete. It is shown that friction constitutes a significant pitfall during such test. In panel tests with continuous simple support, friction between the under-side of the panel and support fixture occurs in two directions, tangentially and radially. The tangential and radial panel movements of the specimen relative to the support have been quantified.

The results show that friction between the concrete panel and the wooden support fixture had a great impact on the maximum load during the test, the residual load and the calculated energy absorption. For the standard test condition, where a (moist) concrete panel is placed directly on a wooden support fixture, the results show that 35% of the apparent energy absorption capacity is due to a friction effect and the remaining 65% is due to fibre action in the concrete panel. Both the test results and the regression analysis show that the coefficient of friction between panel and support fixture increases as the test proceeds.

The wooden support also displays significant wearing and the friction properties may therefore change with the age of the support. The implication is that the use of wooden support in panel tests should not continue!

The CEN square panel test method (EN 14488-5) require support fixture of steel (plus bedding). Apparently, the documentation of the friction effect under such conditions is absent today, and there is consequently a need for documentation of the role of friction in the EN-method, also. The influence of the moisture state of the concrete panel, which may affect the coefficient of friction, should also be elucidated in this regard.

ACKNOWLEDGEMENTS

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REFERENCES

- Bernard E.S. 2005. The Role of Friction in Post-Crack Energy Absorption of Fibre Reinforced Concrete in the Round Panel Test. *Journal of ASTM Int.*, Jan. 2005, Vol. 2, No. 1.
- Bjøntegaard Ø. 2008. Testing of energy absorption for fibre reinforced sprayed concrete. *Proc. of the 5th Int. Symp. on Sprayed Concrete—Modern use of wet sprayed concrete for underground support*. Lillehammer, Norway, 21–24 April 2008, pp. 60–71, ISBN 978-82-8208-005-7. Tekna, Norwegian Concrete Association.
- Bjøntegaard Ø. & Myren S.A. 2008. Energy absorption capacity for fibre reinforced sprayed concrete. Effect of panel geometry and fibre content (Series 2). Technology report no. 2532, Norwegian Public Roads Administration (in Norwegian).
- Bjøntegaard Ø. 2009. Energy absorption capacity for fibre reinforced sprayed concrete. Effect of friction in round and square panel tests with continuous support (Series 4). Technology report no. 2534, Norwegian Public Roads Administration.
- Cook R.A. 1989. Behavior and design of ductile multiple-anchor steel-to-concrete connections. Ph.D thesis, Univ. of Texas at Austin, May 1989.
- DiNoia T.P. & Rieder K.-A. 2004. Toughness of reinforced shotcrete as a function of time, strength development and fibre type according to ASTM C1550-02. *Shotcrete: More Eng. Dev.*—Bernard (ed.), 2004 Taylor & Francis Group London, ISBN 04-1535-898-1, pp. 127–135.
- Funk R.R. 1989. Shear friction mechanisms for support attached to concrete. *Concrete International*, July 1989, pp. 53–58.
- Kurtus R. 2005. Coefficient of Friction Values for Clean Surfaces, www.school-for-champions.com/science/friction_coefficient.htm
- Myren S.A. & Bjøntegaard Ø. 2008. Energy absorption capacity for fibre reinforced sprayed concrete. New test rig, effect of panel geometry and testing laboratory (Series 1). Technology report no. 2531, Norwegian Public Roads Administration (in Norwegian).
- Myren S.A. & Bjøntegaard Ø. 2009. Energy absorption capacity for fibre reinforced sprayed concrete. Effect of panel geometry, fibre content and fibre type (Series 3). Technology report—to be published in 2009.
- Norwegian Concrete Association's publication no. 7 (NB7): Sprayed concrete for rock support. 2003 (in Norwegian).
- NS-EN 12350-6 Testing fresh concrete. Part 6: Density, 2000.
- NS-EN 14488-7 Testing sprayed concrete. Part 7: Fibre content of fibre reinforced concrete. 2006.
- NS-EN 14488-5 Testing sprayed concrete, Part 5. Determination of energy absorption capacity of fibre reinforced slab specimens. 2006.
- Thorenfeldt E. 2006. Fibre reinforced concrete panels. Energy absorption capacity for standard samples. SINTEF memo (in Norwegian).

Evaluating the service life of shotcrete

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ABSTRACT: Over the last decades, extensive research has been conducted to describe and predict the service life of reinforced concrete structures. Several models, some of which are extremely sophisticated, are available today to evaluate and predict the ingress of contaminants into concrete porosity. Unfortunately, the current state of knowledge regarding shotcrete durability, and especially service life prediction, is limited. Criteria have been proposed to relate the quality of in-place shotcrete to its Boiled Water Absorption (BWA). However, these criteria are mostly based on job site observations, and they require more fundamental scientific support. Moreover, the BWA test measures the volume of capillary voids present in concrete, while it is recognized that contaminant ingress, a major durability concern, is mostly caused by diffusion.

The main objective of this research program is to study the relevance of the BWA test to estimate shotcrete durability. Emphasis is placed on shotcrete transport properties. An experimental program was put forward, where 17 shotcrete mixtures (wet and dry-mix) were sprayed in a controlled laboratory environment. The STADIUM™ software is used to model chloride ingress in shotcrete. Results show that BWA alone is not a sufficient parameter to estimate shotcrete service life. It is poorly correlated with diffusion coefficients which are more important in terms of durability.

1 INTRODUCTION

Durability of concrete infrastructure is a growing concern throughout the world. Deterioration mechanisms include mainly freeze-thaw cycling, shrinkage cracking, sulphate attack, and alkali-aggregate reactions. However, in the case of reinforced concrete, the most expensive durability problem is unquestionably associated with chloride-induced corrosion. Steel corrosion results in cracking and spalling of the concrete cover, which leads to further damage caused by amplified ingress of aggressive agents. If the initiation of reinforcement corrosion is detected early, maintenance and repair operations are simple and the associated cost could be minimized. On the other hand, if reinforcement corrosion has reached the point where it has caused significant cracking and spalling of the concrete, repair is more complex and expensive. To avoid this situation, owners and managers often adopt a *service life prediction* approach to anticipate the maintenance operations on their structures to optimize their investment.

Over the last decades, intensive research was conducted to understand how contaminants penetrate into the porosity of cementitious materials and how steel corrosion is initiated within concrete. Numerous models are now available to help infrastructure managers planning rehabilitation operations on their facilities. Those models are based on a thorough scientific approach. They take into account

the environmental conditions in the vicinity of the structure, and the chemical and physical properties of the material. All this research activity not only helped managers in the decision making process, but it also contributed to the understanding of the mechanisms that controls contaminant transport in concrete (or concrete transport properties).

On the other hand, in the shotcrete industry, specifiers often prescribe, in addition to compressive strength, a maximum value of boiled water absorption (BWA). This requirement, initially used to evaluate the quality of the shotcrete placement, has somehow evolved into what many specifiers see as a durability criterion. In the case of conventional cast concrete, this practice was discredited by some authors (De Schutter & Audenaert 2004; Audenaert et al. 2006) because BWA results (which indicate capillary absorption) are poorly correlated with other transport properties (such as ionic diffusion and water permeability). Faced with this situation, a project was undertaken in the Shotcrete Laboratory at Laval University, Quebec, Canada. The objectives behind this study are threefold: generate data on shotcrete transport properties; investigate the influence of mix design/shooting parameters on shotcrete transport properties; and offer guidelines for specifiers to help specify relevant material properties. The article ends with a discussion on the relevance of the BWA test as a durability criterion.

2 RESEARCH SIGNIFICANCE

When shotcrete is specified, for repair or for a new structure, engineers must generally comply with a minimum value of compressive strength (ASTM C1604) and a maximum allowable boiled water absorption (ASTM C642). Contrary to what is sometimes conveyed in the industry, there is no direct relationship between the compressive strength of shotcrete and its BWA. Figure 1 presents BWA results as a function of the compressive strength, obtained from several dry and wet shotcrete mixtures sprayed at Laval University in different projects over the years.

Although a trend is observable, Figure 1 shows that a poor correlation exists between the BWA and the compressive strength. One can observe that between 25 and 45 MPa, absorption values range from 5.3% to 9.1%, which represents a large spread (see next paragraph). It is obvious that the parameters influencing shotcrete BWA are not totally understood and appropriately taken into account. Therefore, in order to comply with the absorption criterion specified, mixture designers

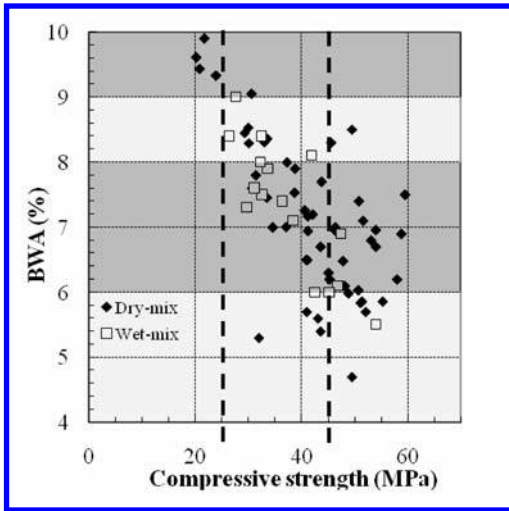


Figure 1. Compressive strength vs BWA.

Table 1. Morgan's Quality Indicators (Morgan et al. 1987).

Sprayed concrete quality	Permeable void volume (%)	Boiled absorption (%)
Excellent	<14	<6
Good	14–17	6–8
Fair	17–19	8–9
Marginal	>19	>9

must often make adjustments based on limited knowledge, which may result in an iterative and expensive process.

The shaded regions presented in Figure 1 reflect the most current state of knowledge concerning shotcrete absorption and durability issues. Indeed, in the 1980s, Morgan et al (1987) proposed qualitative criteria to relate the quality of in-place shotcrete to its boiled water absorption (Table 1).

This classification system is widely used because it is a simple test that relates to the overall quality of the material. Even though these criteria are simple and practical, they do not explain the relationship between the absorption of shotcrete, and its resistance to the penetration of aggressive agents.

3 RESEARCH PROGRAM

A research program was put together to study the transport properties of shotcrete. The project was divided in two parts. In the first part, the influence of mixture design and shooting parameters was investigated. In the second part, the relevance of the BWA test as a durability criterion is discussed.

Twelve shotcrete mixtures (dry and wet-mix) were designed for this project. Mixture variables include the binder composition, the aggregate gradation, and the presence of admixtures and fibres. Three aggregate gradations were used in this project: ACI#1 (mortar), ACI#2 (concrete), and a third identified as MTQ (since it is specified by the Ministry of Transportation of Quebec) which falls between ACI#1 and ACI#2 (see Figure 2).

For the dry process, since the nozzleman adjusts the amount of water added to the mix, the W/Cm ratio is not known. Therefore, as pointed out by Armelin (1997) and Jolin (1999), the material

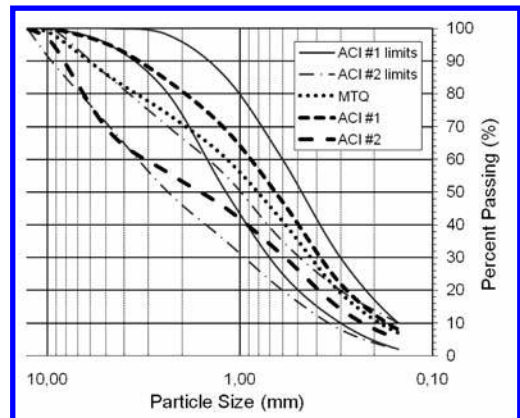


Figure 2. Particle size distributions (limits per ACI 506 (2005)).

consistency is the targeted parameter in dry-mix shotcrete. In this project, several mixtures were sprayed at two consistencies (low and high water content) to represent what is encountered on job sites. All mixtures were sprayed within a practical range of consistencies. The effect of pre-dampening was also studied. Table 2 presents the chemical composition of the cementitious materials used in this project. Tables 3 and 4 present both the *initial* and *in-place* composition of the different mixtures produced in this study. The *in-place* composition was determined following the procedure proposed by Nagi & Whitting (1994). The mixture identification follows this code: the first letter identifies

Table 2. Chemical composition of cementitious materials (%).

Element	Cement 1 Type MS (%)	Cement 2 Type GU (%)	Silica Fume (%)	Fly Ash Type C (%)
CaO	63.2	62.5	1.0	27.7
SiO ₂	20.3	19.7	96.3	32.5
Al ₂ O ₃	4.8	4.6	0.2	19.2
SO ₃	3.4	3.4	0.4	2.8
Fe ₂ O ₃	3.2	3.1	0.1	6.0

Table 3. Initial mix composition.

Mix	Description	Process	Aggregate gradation
CD1	Type GU, 10% silica fume	Dry	MTQ
CD2	Type GU, 10% silica fume, predampened	Dry	MTQ
CD3	Type GU, 10% silica fume Synthetic fibres, powdered AEA	Dry	MTQ
CD4	Type GU, no SCM's	Dry	MTQ
CD5	Type GU, 25% fly ash	Dry	MTQ
MD1	Type GU, 10% silica fume	Dry	ACI#1
MD2	Type GU, 12% silica fume	Dry	ACI#1
MD3	Type GU, 15% silica fume	Dry	ACI#1
MD4	Type GU, 12% silica fume Synthetic fibres, powdered AEA	Dry	ACI#1
CW1	Type MS, 8% silica fume Naphthalene HRWRA 4 ml/kg Cm	Wet	ACI#2
CW2	Type MS, 8% silica fume Naphthalene HRWRA 12 ml/kg Cm	Wet	ACI#2
MW1	Type GU, 12% silica fume Naphthalene HRWRA 10 ml/kg Cm	Wet	ACI#1

Table 4. In-place mix composition.

Mix	W/Cm	Binder content (kg/m ³)	Aggregate content (kg/m ³)	Paste volume (%)
CD1	0,35	511	1604	34,7
	0,54	385	1648	33,4
CD2	0,49	415	1633	34,0
	0,57	392	1608	35,2
CD3	0,37	505	1662	35,3
	0,51	423	1663	35,5
CD4	0,44	437	1703	33,2
CD5	0,41	451	1724	33,4
MD1	0,42	522	1594	39,1
	0,64	395	1596	38,3
MD2	0,44	508	1527	39,1
	0,53	447	1525	38,4
MD3*			N/A	
MD4	0,47	460	1603	36,8
CW1	0,42	405	1744	30,2
CW2	0,47	372	1809	29,6
MW1	0,55	433	1493	37,9

* Results unavailable because of equipment breakdown during the testing operation.

whether the mixture is a concrete (“C”) or a mortar (“M”), and the second letter classifies the process (“D” for dry-mix and “W” for wet-mix). Results for the mixture MD3 are unavailable because of an equipment breakdown during the testing operation. One should observe that out of the 12 initial mixtures (Table 3), some were sprayed at two different consistencies (Table 4). Thus, 17 different *in-place* mixtures were produced in this study.

Shotcreting activities took place in Laval University’s shotcrete laboratory using full-size equipment in a controlled environment. The dry-mix gun used in this project was a rotating barrel ALIVA 246, with a 38 mm (1.5 in) interior diameter hose. The water ring is located 3.0 m (10 ft) upstream from the nozzle. For wet-mix shotcrete, the fresh concrete was pumped using an Allentown Powercreter 10, combined with a 50 mm (2 in.) interior diameter hose, and shot with a ACME nozzle (Figures 3 and 4).

The experimental investigation included the evaluation of the compressive strength (ASTM C1604), the BWA and volume of permeable voids (ASTM C642), and the chloride penetration using the Rapid Chloride Penetration Test (ASTM C1202) and chloride migration test (ASTM C1202 modified). The procedure and theoretical background related to the migration test can be found in the paper by Samson et al (2003). This last experiment is necessary to determine diffusion

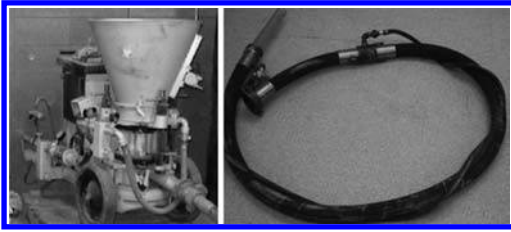


Figure 3. Dry-mix gun and nozzle.



Figure 4. Wet-mix pump and nozzle.

coefficients, a requirement of the service life prediction software STADIUM™. The diffusion coefficient is a critical parameter characterizing the displacement of ionic species into the porosity of porous materials such as concrete.

4 RESULTS

Test results from this project are presented in Table 5 & 6. For clarity purposes, only diffusion coefficients for hydroxyl ions (OH^-) are shown. This choice is arbitrary but it allows for direct comparisons. Moreover, the diffusion coefficients of every ionic species are all related by the same proportional constant, the material tortuosity τ (Samson et al. 2003). This parameter characterizes the intricacy of the path that ions must travel in cementitious materials.

The following sections analyse the results from these tables to better understand what affects BWA in shotcrete.

Table 5. Test results.

Mix	Consistency	UCS* 28d (MPa)	RCPT** (C)	OH^- Diffusion coefficient ($\times 10^{-11} \text{ m}^2/\text{s}$)
CD1	Dry	68,3	431	1,64
	Wet	55,4	922	3,21
CD2	Dry	59,1	830	3,54
	Wet	50,0	1300	4,66
CD3	Dry	52,2	435	1,45
	Wet	39,5	1418	3,48
CD4	In between	48,2	5471	13,60
CD5	In between	53,8	2725	3,76
MD1	Dry	52,5	1119	2,54
	Wet	42,5	2043	5,05
MD2	Dry	75,3	234	0,88
	Wet	54,8	1036	3,51
MD3	In between	53,1	724	2,63
MD4	Wet	51,7	666	1,56
CW1	–	61,4	689	2,69
CW2	–	53,6	713	2,13
MW1	–	53,4	862	2,95

* UCS: Unconfined Compressive Strength.

** RCPT: Rapid Chloride Penetration Test.

Table 6. Test results.

Mix	Consistency	BWA (%)	Volume of permeable Voids (%)	Bulk density (dry) (T/m^3)
CD1	Dry	5,0	11,4	2,29
	Wet	6,8	15,0	2,20
CD2	Dry	6,4	14,2	2,23
	Wet	7,7	16,6	2,17
CD3	Dry	6,0	13,3	2,19
	Wet	7,5	15,9	2,12
CD4	Dry	5,7	13,0	2,26
CD5	Dry	5,7	13,0	2,27
MD1	Dry	7,9	17,1	2,15
	Wet	9,4	19,6	2,08
MD2	Dry	6,5	14,4	2,21
	Wet	8,8	18,6	2,12
MD3	Dry	7,6	16,3	2,15
MD4	Wet	7,2	15,5	2,15
CW1	–	4,8	11,2	2,32
CW2	–	5,1	11,8	2,29
MW1	–	9,1	19,2	2,11

4.1 Boiled water absorption

Previous results from this project (Bolduc et al. 2009) showed that the paste volume has a considerable influence on cast concrete BWA (Figure 5, top). Results also showed that the aggregate grada-

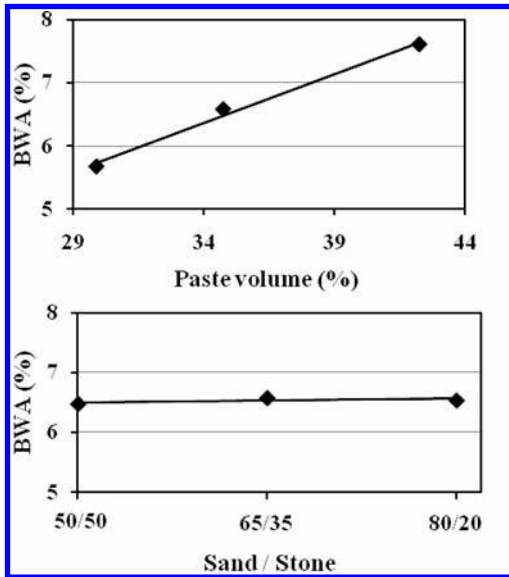


Figure 5. Influence of paste volume (top) and aggregate gradation (bottom) on BWA.

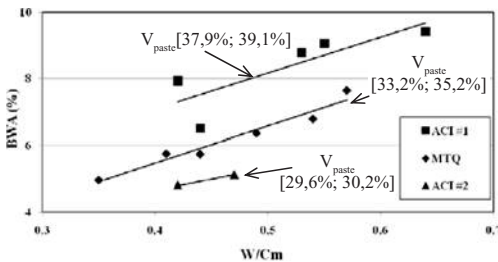


Figure 6. BWA vs W/Cm ratio.

tion alone does not have a significant influence on cast concrete BWA (Figure 5, bottom).

One of the main parameters known to affect the absorption of concrete is the W/Cm ratio. During the hydration process, the relative proportions of water and cementitious materials have a great influence on the capillary porosity. Figure 6 illustrates the relationship between the BWA and W/Cm ratio, for the mixtures sprayed in this project. Results from both dry and wet-mix shotcrete are presented in this graph. However, in this section, results from 13 mixtures are analysed. Indeed, mixtures CD3 and MD4 are not shown on this graph because they are commercial mixtures that contain powdered air entraining admixture and polypropylene fibres, for which exact in-place composition could not be determined. Mixture MD3 is not on the graph either: the W/Cm ratio has not been evaluated because of equipment failure.

At first glance, the correlation between these two parameters is quite poor. However, when the three aggregate gradations used in this project are considered separately (ACI#1, MTQ, and ACI#2), the correlation is increased, as shown in Figure 6. In other words, the graph shows that mortar (ACI#1) leads to higher BWA in comparison with the MTQ gradation, which have a higher BWA compared with ACI#2. One should be careful and keep in mind the results from Figure 5, and not conclude that finer aggregate gradations lead to higher BWA. It was shown in Figure 5 (bottom) that the fineness of the particle size distribution does not have an impact on concrete BWA. Obviously, another step is needed to explain the difference between the three lines in Figure 6.

The rational for these observations is as follows. Figure 5 (top) shows that BWA is directly related to the paste volume. The analysis of results for the shotcrete mixtures is interesting because the range of paste volumes is clearly distinct for the three aggregate gradations (as shown in Figure 6). The mortar (ACI#1) led to paste volumes between 37.9% and 39.1%, MTQ gradation led to values between 33.2% and 35.2%, and ACI#2 brought paste volumes between 29.6% and 30.2% (paste contents expressed here exclude the volume occupied by air bubbles). The conclusion is that it is the *initial aggregate gradation* that has an impact on the in-place paste volume, which has, in turn, an effect on shotcrete BWA. Therefore, two parameters are needed to explain BWA: the in-place paste volume and the paste quality (as expressed by the W/Cm ratio).

4.2 Chloride ingress

The ingress of chloride ions is the root cause related to most reinforced concrete deterioration (Neville 2000). Since the traditional method to evaluate chloride ion penetration (ASTM C1543/ ASTM C1152) is time consuming (ie. ponding for 3 months minimum), the research community has designed over the years accelerated procedures to assess the penetrability of cementitious materials (ASTM C1202). The Rapid Chloride Penetration Test (or RCPT) is a measure of the electrical conductance of concrete that gives an indication of its ability to resist chloride ion penetration. Table 7, presented in ASTM C1202, relates the charge that passed through the sample in 6 hours to the chloride ion penetrability. The higher values denote a higher penetrability, and therefore an expected reduced durability.

It is well known in the literature that chloride ion penetrability is influenced by the W/Cm ratio. Figure 7 presents the relationship between RCPT

Table 7. ASTM C1202 classification system.

Charge passed (C)	Chloride ion penetrability
>4000	High
2000–4000	Moderate
1000–2000	Low
100–1000	Very low
<100	Negligible

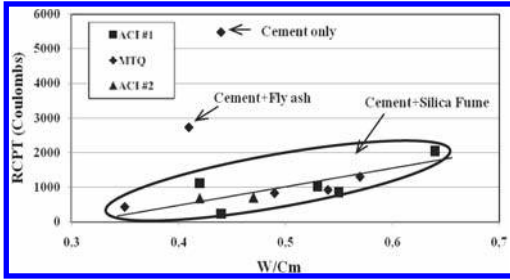


Figure 7. RCPT vs W/Cm ratio.

results and W/Cm ratios for the shotcrete mixtures sprayed in this project.

Two conclusions can be drawn from this graph. The first one is that contrary to the BWA test, there is no distinction between the three aggregate gradations/paste contents. The binder content is obviously more significant. The second one is that RCPT and W/Cm ratios are poorly correlated. However, observation of the mixture designs shows that the two points higher than 2500 coulombs do not contain silica fume. Excluding these two points, the correlation is much better. This graph reveals the strong influence of silica fume on the electrical conductivity of cementitious materials.

In the presence of silica fume, the analysis of RCPT results can be misleading. As pointed out by Torii & Kawamura (1994), the presence of silica fume leads to a decrease in the hydroxyl ion concentration in the pore solution, which reduces the overall electrical conductivity of the material. Therefore, RCPT results underestimate the real chloride ion penetrability when silica fume is used in the mixture. This is the reason why the modern approach is to estimate diffusion coefficients to predict the chloride ingress. Just as for the BWA test, the RCPT is not adequate, by itself, to properly describe the penetrability of shotcrete.

5 DISCUSSION

This section presents a discussion on the relevance of using the BWA test as a durability criterion. Here, the term *durability criterion* only refers to the ingress of contaminants into the shotcrete porosity

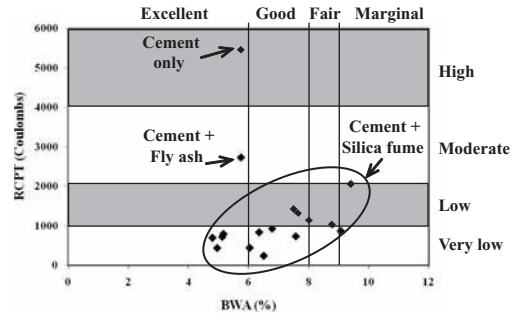


Figure 8. Relationship between RCPT and BWA.

and excludes other deterioration mechanisms. The main objective of this section is to determine if the BWA is pertinent to estimate chloride penetration. The first step taken here is to observe the correlation between RCPT and BWA (Figure 8).

The classification system proposed by Morgan (1987), where the BWA is related to the overall shotcrete quality, is shown on the graph. The regions identified on the right-hand side indicate the chloride penetrability, as presented in the ASTM C1202 standard. Again, the correlation between these parameters is low. This can be explained by the fact that both properties are not similarly affected by changes in the porosity. As pointed out by Sato & Agopyan (2001), there is a difference between the total volume of pores and pore dimension. While the BWA is more affected by the size of the *accessible* pores in the concrete surface, the RCPT test will be most affected by the connectivity of the pores, and a lot less by their size. In terms of chloride ion transport, both size and connectivity of the porous system in the concrete are considered to have a significant influence.

Two important observations can be made from Figure 8. First, if only the mixtures with *Very low* penetrability are considered (lower strip on the graph), the BWA classification ranges from *Marginal* to *Excellent*. On the other hand, if only the mixtures classified as *Excellent* are analyzed, the chloride penetrability ranges from *Very low* to *High* as soon as the binder type is changed. Even if only two mixtures were tested without silica fume, the authors believe the results show the obvious influence of the binder composition. These observations are very important because, depending on the environmental condition surrounding the shotcrete application, two situations can occur:

- High-quality shotcrete (*Very low* penetrability) can be rejected because it does not meet the absorption criterion specified, or
- Poor quality shotcrete (*High* penetrability) might be accepted because it presents an adequately low BWA value.

It is believed that both cases present a poor assessment of the real material quality/durability. Owners and specifiers should consider a larger range of parameters when selecting material specifications such as the environmental exposure, the type of structure considered (above or below ground, etc.) and particularly previous performance records for a given mixture design.

6 CONCLUSION

Based on an extended experimental research program where 17 shotcrete mixtures (wet- and dry-mix) were sprayed and characterized, the following conclusions can be drawn.

The *initial* aggregate gradation has a direct impact on the *in-place paste volume*, which is, in turn, a parameter that affects significantly the shotcrete boiled water absorption. This observation is valid for both wet- and dry-process shotcrete.

Rapid chloride penetration test results are greatly influenced by the presence of silica fume, and to a lesser extent, by the presence of fly ash. Supplementary cementing materials do not seem to affect significantly the shotcrete boiled water absorption.

The boiled water absorption test alone is not a reliable parameter for estimating shotcrete durability, such as chloride-induced corrosion. This test is poorly correlated with RCPT and diffusion coefficients, which are more relevant in terms of chloride transport.

A more thorough approach is needed to understand all the implications of the shotcrete placement process on the durability of the resulting in-place concrete. The effect of parameters such as set accelerators, material velocity or the presence of compaction voids are not yet fully understood in terms of resulting in-place composition and significance to the long-term durability of the shotcrete. More effort is needed to better understand the pneumatic placement process itself and also to actually model and predict the long-term behavior of shotcrete exposed to harsh environments.

REFERENCES

- ACI-506R-05. 2005. *Guide to Shotcrete*, ACI International.
- Armelin, H.S. 1997. *Rebound and toughening mechanisms in steel fiber reinforced dry-mix shotcrete*, Ph.D. thesis, University of British Columbia.
- ASTM International 2002. C1543—Standard Test Method for Determining the Penetration of Chloride Ion into Concrete by Ponding, Annual Book of ASTM Standards, Concrete and aggregates, Vol. 04.02, Philadelphia.
- ASTM International 2004. C1152—Standard Test Method for Acid-Soluble Chloride in Mortar and Concrete, Annual Book of ASTM Standards, Concrete and aggregates, Vol. 04.02, Philadelphia.
- ASTM International 2005. C1202—Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration, Annual Book of ASTM Standards, Concrete and aggregates, Vol. 04.02, Philadelphia.
- ASTM International 2005. C1604—Standard Test Method for Obtaining and Testing Drilled Cores of Shotcrete, Annual Book of ASTM Standards, Concrete and aggregates, Vol. 04.02, Philadelphia.
- ASTM International 2006. C642—Standard Test Method for Density, Absorption, and Voids in Hardened Concrete, Annual Book of ASTM Standards, Concrete and aggregates, Vol. 04.02, Philadelphia.
- Audenaert, K., Boel, V. & Schutter, G.D. 2006. "Influence of Capillary Porosity on the Transport Properties of Self Compacting Concrete", *Eighth CANMET/ACI International Conference on Recent Advances in Concrete Technology*.
- Bolduc, L.-S., Jolin, M. & Bissonnette, B. 2009. *Évaluation de la performance du béton projeté*, (RF)2B, ENS Cachan, France, in French.
- De Schutter, G. & Audenaert, K. 2004. "Evaluation of water absorption of concrete as a measure for resistance against carbonation and chloride migration", *Materials and Structures*, **37**(9), 591–596.
- Jolin, M. 1999. *Mechanism of placement and stability of dry process shotcrete*, Ph.D. thesis, University of British Columbia, Vancouver.
- Morgan, D.R., McAskill, N., Neill, J. & Duke, N.F. 1987. "Evaluation of silica fume shotcrete", *International Workshop on Condensed Silica Fume in Concrete*, Montreal.
- Nagi, M. & Whiting, D. 1994. "Determination of Water Content of Fresh Concrete Using a Microwave Oven", *Cement, Concrete and Aggregates*, **16**(2), 125–131.
- Neville, A.M. 2000. *Properties of Concrete*, Édition Eyrolles, Paris, 806 p.
- Samson, E., Marchand, J. & Snyder, K. 2003. "Calculation of ionic diffusion coefficients on the basis of migration test results", *Materials and Structures*, **36**(3), 156–165.
- Sato, N.M. & Agopyan, V. 2001. "Influence of porosity on water and chloride ion transport through concrete", *Third International Conference on Concrete Under Severe Conditions*, Vancouver, Canada, 412–419.
- Torii, K. & Kawamura, M. 1994. "Pore structure and chloride ion permeability of mortars containing silica fume", *Cement and Concrete Composites*, **16**(4), 279–286.

The use of shear wave velocity for assessing strength development in Fibre Reinforced Shotcrete

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ABSTRACT: This paper presents the results of a laboratory study where the relationship between the travel velocity of shear waves V_s through concrete and the unconfined compressive strength UCS of the material has been investigated. Over the range of conditions addressed in the work, a unique relationship was found between UCS and V_s^2 . Based on this result, it is suggested that, after establishing the relationship between UCS and V_s for a particular site, non-destructive shear wave velocity measurements can be used to assess the quality of Fibre Reinforced Shotcrete (FRS) in the field. This risk management approach has the potential to identify whether the FRS target strength has been achieved after the specified curing period, but also provides an opportunity to accelerate productivity (through earlier re-entry or filling) if the material reaches the specified strength prior to the established curing period.

1 INTRODUCTION

Fibre Reinforced Shotcrete (FRS) has found in recent years many applications in underground construction, especially in the mining industry (Yoggy 2005). The application of FRS in a mining environment provides considerable benefits in terms of accelerating development turn-around times and rapid/safe fill bulkhead construction (Methven 2004). Successful implementation of this technology in the mining industry is dependent on obtaining consistent high strengths at very early hydration periods, which, based on quality control data from a number of different sites, is difficult to achieve. Reasons for the high variability in FRS strengths at early age may be a consequence of logistical difficulties, rugged working conditions, chemistry problems, dosing problems, and variable environmental conditions.

FRS is usually mixed until all components (including sand, aggregates, water, fibres and additives) are added, and it continues to mix from the batching site to the worksite. Delays in delivery to the work site, which occur regularly in a mining environment, can cause the material to become unusable (meaning that the load must be rejected), but water and/or chemical additives can be added (in often uncontrolled doses) to prolong the setting time. While this provides a usable product, the resulting FRS strength is obviously reduced. Besides water and admixture incorporation during

transportation, variability in the final product might also occur at the work site due to variability of set accelerator addition at the nozzle. Furthermore, in underground settings the environmental curing conditions can vary considerably and it is well known that this has a significant influence on the strength development of FRS.

To illustrate the influence of one of the above mentioned variables (i.e. mix design, curing conditions, hydration time, etc), the results of a series of unconfined compressive strength (f_c) tests presented by Yi et al. (2003) are reproduced in Figure 1. This figure presents f_c for concrete mixes containing different water cement ratios (w/c) after different periods of hydration. The hydration times presented in Figure 1 are representative of those adopted for design in the mining industry. Figure 1 clearly illustrates that the strength achieved after the period of interest of this study (1–3 days) is extremely sensitive to the w/c ratio, creating very significant changes in the resulting strength.

While only the influence of variations in w/c has been presented, the influence of the other variables on early age strength is expected to be similar. This is because the early age strength is highly sensitive to the initial hydrate growth, which can be significantly influenced by each of the mentioned factors. Because of the significant consequences associated with the inadequate FRS strength and the high likelihood of variability in the strengths achieved at early age, FRS mixes are, in most cases,

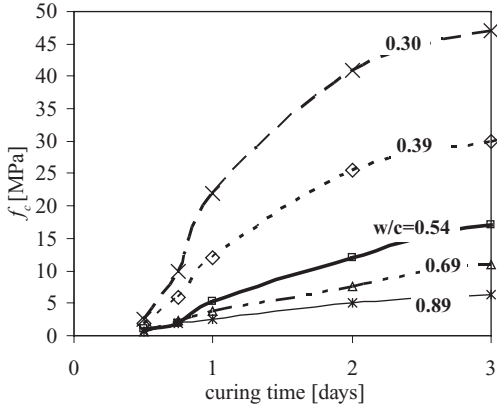


Figure 1. Compressive stress-strain curve of concrete at 5 ages with different water/cement ratios (Yi et al. 2003).

over-designed meaning that on many occasions re-entry times (or backfilling times) could actually be earlier. But, without a rigorous technique for checking FRS quality prior to re-entry, overdesign is necessary to manage the risk of variability.

This paper focuses on a seismic method for early age FRS QC testing, which the authors believe could provide a means of assessing the development of strength with time in a mining environment. Such a management system would allow operators to take advantage of situations where the target strength is achieved earlier than the design re-entry period, while ensuring that personnel are not exposed to unfavorable conditions where the design strength has not been achieved even after the design re-entry period.

Seismic methods have been used for a long time to characterize setting and hardening of cementitious materials (Sayers & Grenfell 1993, De Belie et al. 2005, Boumiz et al. 1996, Rapoport et al. 2000, Voigt et al. 2003, Akkaya et al. 2003). The method being investigated in this document is based on measuring the shear wave velocity (V_s) of a FRS specimen (beam) that has been cast and cured at the location of spraying. As V_s is physically related to the small strain shear stiffness (G_0) this provides a method for measuring the evolution of G_0 with time.

Since FRS design typically targets a specific strength (f_c) (rather than the small strain shear stiffness), prior to the implementation of the proposed technology (for quality control testing purposes), it is important to ensure that G_0 (or V_s) can be consistently related to f_c over the range of conditions that are likely to vary in a typical FRS situation. Therefore, this paper presents an experimental study where the strength of a given FRS mix is varied by altering the curing period, water content, cement content and curing environment

to determine if a unique relationship exists across all of these variables.

While a number of QC systems have been suggested in the past, the proposed technique is considered attractive because: 1) the testing method is non-destructive; 2) the testing equipment is mobile and can therefore be easily transported to a specimen curing in the field; 3) the shear wave velocity (V_s) is physically related to the small strain shear stiffness, hence the technique measures the evolution of a true mechanical property of the hardening material (see Section 2).

2 PRINCIPLES AND BACKGROUND

2.1 Principles of waves

Waves traveling in one direction, but with particle motion along another perpendicular direction, are defined as shear waves. Under the assumption that the shear wave travels in an infinitive homogenous medium, the shear wave velocity (V_s) is equal to:

$$V_s = \sqrt{\frac{G_0}{\rho}} \quad (1)$$

where G_0 is the elastic small strain shear modulus and ρ is the mass density of the medium (Santamarina et al. 2001). For i) wavelengths larger than the size of heterogeneities of the material (i.e. size of aggregate in concrete), ii) short travel paths, and iii) wavelengths larger than the minimum lateral dimension of the sample, attenuation and losses can be neglected and Equation 1 is valid.

The reason why shear waves are preferred to compressive waves is because shear waves are independent of the presence of pore water while compression wave velocities can be influenced by the high compressive stiffness of the water phase. This is particularly relevant in early age FRS where the material is expected to possess a high water content. In the early stages of hydration, concrete behaves as a medium with very little shear strength and stiffness ($G_0 \approx 0$), hence no shear waves are transmitted. But after that a continuous path of connected grains is created through the paste due to the hydration process the velocity of shear waves increases rapidly (Boumiz et al. 1996).

2.2 Relationship between elastic shear modulus and compressive strength

From the theory of continuum mechanics, it is known that:

$$G_0 = \frac{E}{2(1+\nu)} \quad (2)$$

where E is Young's modulus and ν the Poisson's ratio of the material. The mechanical characteristics of concrete vary with the degree of hydration, which is a function of time, components present in the mix and temperature and humidity at curing conditions (Lawrence et al. 1977). Many relationships between compressive strength and Young's modulus exist in literature, but most of them only consider how these characteristics vary with time for a given mix. Based on experimental results on mortar samples, Boumiz et al. (2001) presented a Young's modulus vs. degree of hydration and compressive strength vs. degree of hydration plot. This plot is reproduced in Figure 2.

Combining the results presented in Figure 2, Figure 3 presents the Young's modulus vs. compressive strength relationship, for degree of hydration between 0.2 and 0.6.

The data presented in Figure 3 indicates a clear trend for the development of strength and Young's modulus with time. A similar relationship

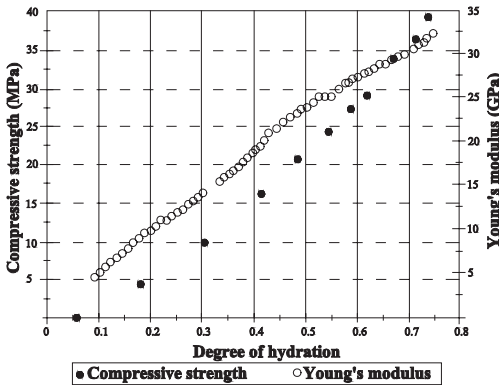


Figure 2. Compressive strength and Young's modulus as functions of degree of hydration (Boumiz et al. 2001).

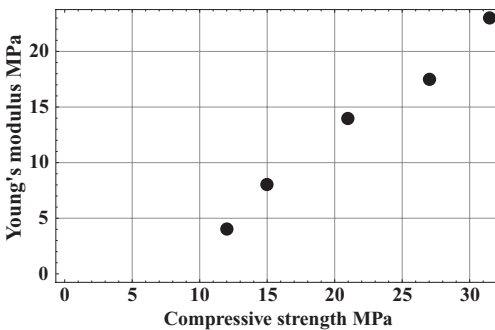


Figure 3. Young's modulus vs. compressive strength relationship of mortar samples for different degrees of hydration.

would therefore be expected between compressive strength and shear modulus, according to Equation 2. It should be noted that Poisson's ratio also varies with the degree of hydration of the material, and this may affect the shape of the relationship between G_0 and f_c .

3 EXPERIMENTAL METHODS AND PROCEDURES

3.1 Testing procedure

The aim of this study was to determine if, over the range of variables that are likely to create a change in FRS strength, a unique relationship exists between V_s and f_c during the early stages of hydration. For the purpose of these experiments, a fibre reinforced concrete mix design was used without any chemical admixture. The base case concrete mix design is reported in Table 1. Concrete was cast into $165 \times 75 \times 75$ mm card-board boxes. This specimen shape is similar to the one proposed by Morgan (1998) for the beam end test. In that test, FRS is sprayed into an open-ended mould.

For each beam V_s is measured before the sample is tested to failure in unconfined compression. During compression testing, blocks of 75×75 mm soft wood are placed above and below the end of the sample to be tested so that the load is uniformly applied on a square surface. In this way, the obtained compressive strength is closer to the value of cubic than cylindrical samples. According to the Australian Standards (AS3600, 2001), the compressive strength refers to cylindrical samples of 100 mm diameter and 200 mm height, and a shape factor must be considered for different shapes (typically, the compressive strength of cubic samples is 25% higher than the strength of cylinders (del Viso et al. 2007)). In this study, f_c refers to the compressive strength of beam ends, and no shape factor is applied.

3.2 Measurement of shear wave velocity

The experimental setup for the measurement of V_s consists of the following equipment: a signal generator outputting a pulsed waveform, an

Table 1. Concrete mix design.

Components	Kg/m ³
Fine aggregate (<4 mm)	805
Sand (grade 45/50)	765
Cement (high early)	389
Water	283
RAD 55 Macro-synthetic Fibres	6.75

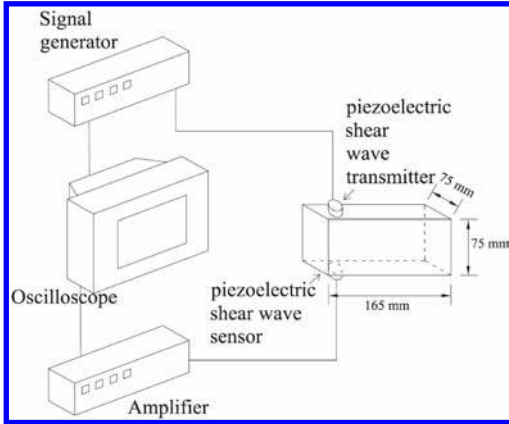


Figure 4. Schematic diagram of the shear wave testing equipment.

oscilloscope, a piezoceramic shear wave transmitter, a piezoelectric shear wave sensor, and an amplifier. The schematic diagram of the equipment is shown in Figure 4.

The system delay time, due to cabling and equipment, is measured by placing the transmitter and receiver directly together and measuring the arrival time of the received signal. This delay is subtracted from all readings. For good results the transmitter and receiver are properly coupled to the sample through a layer of petroleum jelly.

3.3 Experimental program

As stated in the introduction, the variability of the strength of a specific mix design depends on different factors, amongst them: uncontrolled addition of water and chemical admixture during transportation, temperature and humidity conditions at curing site and, of course, time (crucial factor for early age concrete as shown in Figure 1).

Table 2 presents a summary of the mix variations considered, hydration conditions and hydration periods for each of the different scenarios considered in this test work program.

It should be noted that curing times shorter than 5.5 hours couldn't be used for the specific used mix design (without accelerants and superplasticizers). Concrete specimens younger than 5.5 hours were too soft to be crushed or even removed from the mould.

The first series, Dt, were cast to investigate the $f_c - V_s^2$ relationship for different hydration times ranging from 5.5 to 12 hours. Series Dc involved changes in the cement addition (from the base case mix) by +20% to -10%, to investigate the influence on the $f_c - V_s^2$ relationship. Series Dw involved modifying the water content (by +10% and -20%)

Table 2. Concrete specimens used in the experiments.

Series	Mix	Curing time [hours]	Environmental conditions
Dt-1	Table 1	5.5	Ambient
Dt-2	Table 1	6.5	Ambient
Dt-3	Table 1	7.5	Ambient
Dt-4	Table 1	9.5	Ambient
Dt-5	Table 1	12	Ambient
Dc-1	Table 1	12	Humidity chamber (21°C)
Dc-2	Table 1 + 20% cement	12	Humidity chamber (21°C)
Dc-3	Table 1 - 10% cement	12	Humidity chamber (21°C)
Dw-1	Table 1	12	Humidity chamber (21°C)
Dw-2	Table 1 + 10% water	12	Humidity chamber (21°C)
Dw-3	Table 1 - 20% water	12	Humidity chamber (21°C)
De-1	Table 1	12	Fridge (4°C)
De-2	Table 1	12	Oven (50°C)
De-3	Table 1	12	Oven (100°C)

to assess the influence on the $f_c - V_s^2$ relationship. Finally, series De was cast using the base case mix, but these specimens were cured under different environmental conditions, which included curing temperatures of 4°C, 50°C and 100°C to assess the influence of curing environment on the $f_c - V_s^2$ relationship. Two beams per series were cast for each condition making a total of 28 beams.

4 RESULTS

The measured strengths are plotted as f_c against hydration time in Figure 5. This figure illustrates that the variations in mix design and curing conditions, which have been considered in this series of experiments, leads to a significant variation in material strength. To illustrate the ability of V_s to capture this variability each of the measured f_c values are plotted against V_s^2 in Figure 6.

The dashed line in Figure 6 represents the best fit relationship between V_s^2 and f_c . This relationship appears to have a reasonable fit to the presented data with the square of the error over the entire data set being 0.96.

The encouraging experimental results provide confidence that any changes in material strength, which are a consequence of the considered

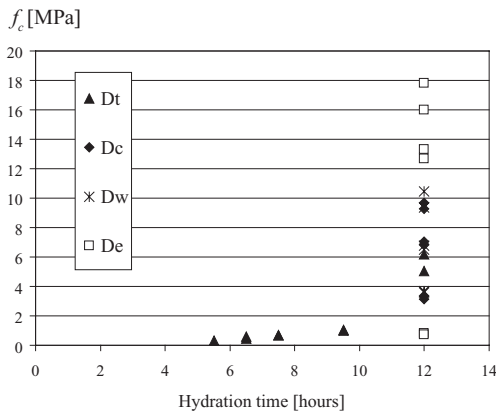


Figure 5. Unconfined compressive strength versus time for the experimental series reported in Table 2.

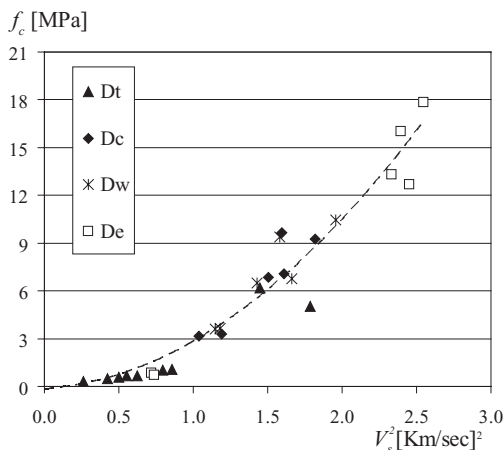


Figure 6. Unconfined compressive strength versus square velocity of shear waves, for the experimental series reported in Table 2.

variables, can be appropriately detected through changes in the shear wave velocity. Furthermore, once a relationship has been developed for a particular site, quantitative criteria (in terms of a required shear wave velocity) can then be established to manage re-entry times or appropriate times to commence filling behind FRS bulkheads. For quality control purposes it may be more appropriate to select a lower-bound criterion on the presented data (rather than the best fit). To provide an example of this application, using the relationship presented in Figure 6, if targeting 2 MPa for development re-entry one might set a V_s criterion of >1 km/s, or given a sample thickness of 75 mm a travel time of <75 μ s.

5 CONCLUSION

This paper has presented shear wave velocity measurements as an effective method for quality control testing of early age strength for FRS. This method is considered to be advantageous because it is non-destructive, the equipment is easily transportable, and it measures a true mechanical characteristic of the material.

Testing illustrated that, for a given base mix design, a unique relationship exists between the square of the shear wave velocity and the unconfined compressive strength when either hydration time, water content, cement contents or curing conditions are varied. This provides confidence that the technique could identify reduced strengths that have resulted from variations in any of these characteristics, but also identify when a particular FRS has reached the target strength and is therefore suitable for re-entry (or filling behind a bulkhead).

While the presented results are encouraging, prior to the implementation of such technology additional verification studies are considered necessary to assess if the unique relationship is valid for other characteristics that might change the hydration properties of FRS, such as admixture addition. It is also crucial to further investigate the correlation for very early age samples (curing times between 1 and 5 hours), and to obtain this it is necessary the use of accelerants and superplasticizers in the mix.

In addition to verifying the technical applicability of the proposed methodology the authors are also investigating a number of operational aspects that must be addressed such as protection of the relevant equipment and ease of use for underground operational personnel.

REFERENCES

- Australian Standard, 2001. AS3600 *Concrete structures*, Standards Australia International, Australia.
- Akkaya, Y., Voigt, T. Subramaniam, K.V. & Shah, S.P. 2003. Nondestructive measurement of concrete strength gain by an ultrasonic wave reflection method, *Materials and Structures* (36): 507–514.
- Boumiz, A., Vernet, C. & Cohen Tenoudji, F. 1996. Mechanical properties of cement pastes and mortars at early ages, *Journal of Advanced Cement-Based Materials* (3): 94–106.
- De Belie, N., Grosse, C.U., Kurz, J. & Reinhardt, H.W. 2005. Ultrasound monitoring of the influence of different accelerating admixtures and cement types for shotcrete on setting and hardening behaviour, *Cement and Concrete Research* (35): 2087–2094.
- del Viso, J.R., Carmona, J.R. & Ruiz, G. 2007. Shape and size effect on the compressive strength of high strength concrete. *Cement and Concrete Research* (38): 386–395.

- Lawrence, F.V., Young, J.F. & Berger, R.L. 1977. Hydration and properties of calcium silica pastes, *Cement and Concrete Research* (7)4: 369–377.
- Methven, G. 2004. In-cycle application of fibre reinforced shotcrete, *Shotcrete: More Engineering Developments-Bernard (ed.)* Taylor & Francis Group, London: 193–200.
- Morgan, D.R. 1998. Steel fibre reinforced shotcrete for underground support: civil applications. *Australian Shotcrete Conference, Sydney, Australia*, 8–9 October 1998: 1–18.
- Neville, A.M. 1981. *Properties of Concrete*. Pitman Publishing, London.
- Rapoport, J., Popovics, J.S., Subramaniam, S.V. & Shah S.P. 2000. Using ultrasound to monitor stiffening process of concrete with admixtures, *ACI Materials Journal* (97)6: 675–683.
- Santamarina, J.C., Klein, K.A. & Fam, M.A. 2001. *Soils and Waves: Particulate Materials Behavior, Characterization and Process Monitoring*. Wiley.
- Sayers, C.M. & Grenfell, R.L. 1993. Ultrasonic propagation through hydrating cements, *Ultrasonic* (31)3: 147–153.
- Simms, P. & Grabinsky, M. 2009. Direct measurement of matric suction in triaxial tests on early-age cemented paste backfill. *Canadian Geotechnical Journal* (46)1: 93–101.
- Voigt, T., Akkaya, Y. & Shah, S.P. 2003. Determination of early age mortar and concrete strength by ultrasonic wave reflections, *Journal of Materials in Civil Engineering* (15)3: 247–253.
- Yi, S., Kim, J. & Oh, T. 2003. Effect of strength and age on stress-strain curves of concrete specimens, *Cement and Concrete Research* (33): 1235–1244.
- Yoggy, G.D. 2005. The History of Shotcrete. Part 1, 2 and 3, *Shotcrete* (Summer): 26–32.

Shotcrete tunnel linings with steel ribs: Stress redistribution due to creep and shrinkage effects

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ABSTRACT: To study the time-dependent behaviour of sprayed concrete linings coupled to steel ribs, an accurate monitoring system was developed in an experimental tunnel including strain gauges, pressure cells, incremental extensometers and topographic controls. The significant delayed deformation under almost constant load, reaching 5–6 times the initial elastic strain, and the stress redistribution from shotcrete to steel ribs, loaded up to four times their initial value in a few weeks, were the most interesting results due to creep and shrinkage effects. The elastic modulus and the creep deformation of shotcrete with different dosages of accelerators were investigated, based on the in situ data readings and through specific laboratory tests on samples collected at site. Using creep parameters from the experimental evidence, a simplified calculation approach, based on the linear algebraic Age-Adjusted Effective Modulus Method (AAEMM), was developed in order to simulate the stress and strain variations over time. A comparison with the creep parameters by CEB-FIP Model Code 1990 for ordinary concretes was also carried out.

1 INTRODUCTION

1.1 Project overview

In Italy the use of wet mix shotcrete in road tunnels under weak rock is currently intended only for temporary support, usually combined with steel ribs. Final permanent linings are cast in situ with thick sections made using reinforced or plain concrete, depending on how significant the expected bending moment is.

In order to obtain a cost-saving solution for service tunnels (by pass or safety tunnels), the possibility to use shotcrete not only as the primary lining (ie. as temporary support) but also as the secondary permanent lining, instead of cast in situ concrete, has been recently considered. To this end, the usual primary lining, formed by one shotcrete layer coupled to steel ribs, is improved by a second shotcrete layer with an intermediate waterproofing sprayed-on membrane aimed to avoid water ingress. Whereas the first shotcrete layer is sprayed just after having placed the steel rib at each excavation step, the second shotcrete layer is sprayed later, after the waterproofing membrane, far away from the excavation face. As with the so-called “single shell” approach (Golser and Kienberger 1997) it can be assumed the two shotcrete layers act together but as a composite structure.

1.2 Research objectives

As in the rest of the Italian construction industry the use of shotcrete is mainly intended for provisional or non-structural works. Dimensioning is performed with very simple calculations methods, often suitable for ordinary concrete structures, nevertheless leading to estimated values of stresses and deformations that are too rough for a permanent load-bearing structure.

Unlike when it is simply considered as temporary, using shotcrete lining as permanent support requires a better and deeper understanding of its time-dependent behaviour in order to achieve a more efficient and reliable design. The current research program, presented in this paper, was intended to overcome our current lack of case studies and practical experience related to creep and shrinkage of shotcrete, and the almost total absence of specific calculation rules for structural shotcrete in the national codes including the definition of the constitutive law and the time-dependent behaviour.

An experimental, 60 metre long and 9 metre diameter by-pass tunnel has been excavated adopting a double layer shotcrete lining with steel ribs in the first layer and an intermediate waterproofing sprayed-on membrane between the first and the second layer. Two different products were tested as waterproofing in order to compare the different behaviour in

allowing shear stress transfer. This particular issue is currently being studied and some tests have already been held while others are still planned in order to investigate the specific constitutive laws related to the membrane compound and its bonding capacity with sprayed concrete, and so be able to define the whole system working as a sandwich of shotcrete—waterproofing membrane—shotcrete.

A proper monitoring system was provided in the lining of the trial tunnel in order to check strain and stresses in the two shotcrete layers and in the steel ribs. Shotcrete samples were also taken on site and different laboratory tests were performed in order to deepen our knowledge regarding the Modulus of Elasticity, shrinkage and creep behaviour of the adopted shotcrete, leading to an improved design approach. The monitoring system allowed us to understand the limits of the ‘simplified approach’ often used by designers in which the temporary shotcrete lining, reinforced with steel ribs, is assumed to behave as an ordinary reinforced concrete structure. The importance of the time-varying behaviour of shotcrete in the computation of stresses within the lining and the differences compared to reinforced concrete were confirmed: a load transfer from shotcrete to steel ribs was shown by data readings, explainable only by shrinkage and creep effects. The development of a calculation method was made possible by introducing the parameters derived from experimentation for the specific shotcrete, adjusting the typical formulation of linear algebraic law of viscosity, already consolidated for cast concrete and also mentioned in the Italian and European Codes.

1.3 Geology and characteristics of the trial tunnel

The rock formation consists of siltstone with some limestone inclusions characterized by an average unconfined compressive strength ranging between 35 and 40 MPa (determined by means of a Schmidt hammer) and a GSI ranging between 40 and 45 MPa.

The trial tunnel was realized through a full-face excavation, mechanically performed at a ranging

depth between 240 and 280 m. The internal environmental characteristics during the tunnel construction included temperatures in the range of 18–20°C and dry conditions (estimated relative humidity of about 50–60%) due to the presence of continuous forced ventilation and the complete absence of water seepage.

The lining consisted of a couple of double-T steel ribs (IPN180 profile) each 1.2 m, a first 20 cm thick shotcrete layer, reinforced with hooked end steel fibres (33 mm length, 0.55 mm diameter) and a second 10 cm thick shotcrete layer. Wet-mix shotcrete was used with the admixtures detailed in Table 1.

2 MONITORING SYSTEM DESCRIPTION

The monitoring system was designed to acquire information regarding both tunnel displacements and stresses in the two shotcrete linings. As for

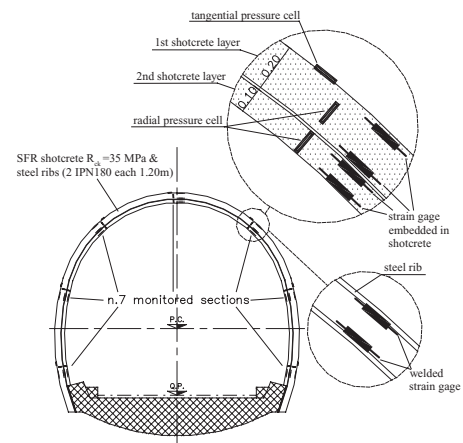


Figure 1. Typical monitoring station.

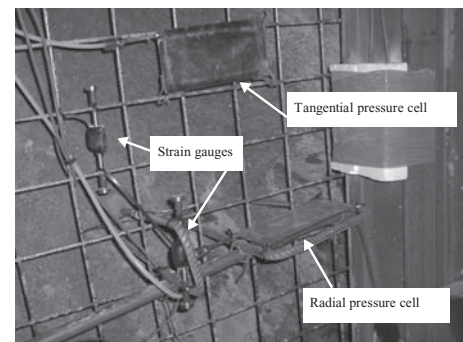


Figure 2. Typical monitoring station.

Table 1. Shotcrete mix design.

Characteristic compressive strength (MPa)	C28/35
Water/cement ratio	0.42
Cement (CEM IV/A-P 42.5R)	450 kg/m ³
Silica fume	25 kg/m ³
Superplasticizer	7.2 kg/m ³
Accelerator	4–8%
Sand < 2 mm	150 kg/m ³
Sand < 6 mm	970 kg/m ³
Crushed stone 5 < ϕ < 10 mm	510 kg/m ³
Steel fibers (1st shotcrete layer only)	30 kg/m ³

tunnel convergence and settlement, convergence stations on the steel ribs involving 5 topographic targets on each, and radial incremental extensometers, were installed. In order to estimate strains and stresses in the lining, monitoring stations were installed (Figures 1 & 2) that included: 14 strain gauges on steel ribs, installed in couples (one on the external edge one on the internal one) along each rib in order to estimate the stress in the steel ribs; 7 tangential pressure cells between the rock and the primary shotcrete layer in order to estimate the contact pressure on the lining; 7 radial pressure cells in the primary shotcrete layer in order to estimate the axial force in the primary shotcrete layer; 7 pairs of strain gauges in the primary shotcrete layer in order to estimate the strain distribution within the primary shotcrete layer; 7 radial pressure cells in the secondary shotcrete layer in order to estimate the axial force; and 7 pairs of strain gauges in the secondary shotcrete layer in order to estimate the strain distribution. All the 63 instruments were cabled to a data acquisition and recording system in order to obtain monitoring data in real time.

3 DESIGN LOADS

During design, both a simplified approach (confinement-convergence method) and 2D and 3D Finite Difference (FLAC) analyses were performed in order to get an estimation of the loads expected during and after the construction of the by-pass. The values of the radial pressures around the tunnel obtained by the different types of analysis ranged between 300 and 360 kPa. Hence, assuming a radial pressure equal to 330 kPa, a rough estimation of the axial force in the lining can be derived by the Boile-Mariotte formula:

$$N = p \cdot D/2 = 1490 \text{ kN/m} \quad (1)$$

where D = tunnel diameter and p = radial pressure.

4 MONITORING SYSTEM RESULTS

The monitoring system allowed us to record time-variations of siltstone pressure against the lining, strain distribution in the steel ribs, strain and average stress distribution in the shotcrete layers. The following are the main results of these measurements.

Only one tangential pressure cell worked properly and recorded time-dependent magnitudes of the siltstone pressure varying between 250 and 300 kPa; all the others cells recorded time-dependent values decreasing to zero. This well-known behaviour of the pressure cells, due to both

loss of contact with the surrounding material and the arching effect in the rock resulting in progressive unloading, was confirmed by the necessity to re-pressurize the cells a few days after their installation (see Fig. 13).

Strain gauges in the steel ribs recorded compressive strains between 1400 and 2700 $\mu\epsilon$ at the crown, revealing stresses of 280–300 MPa close to yielding, and compressive strains of 600–800 $\mu\epsilon$ at the shoulders, corresponding to stresses between 130 and 180 MPa (see Fig. 11). Radial pressure cells in the 1st shotcrete layer recorded mean stresses within the range 1.0 to 1.7 MPa in the upper part of the section and lower values (within the range 0 to 0.5 MPa) near the footings (see Fig. 3). Strain gauges in the 1st shotcrete layer recorded a prevailing compressive strain with values in the range 900 to 1700 $\mu\epsilon$ in the upper part and 0 to 500 $\mu\epsilon$ near the footings (see Fig. 4). Radial pressure cells in the 2nd shotcrete layer mainly recorded a mean stress of 35–55 kPa except for one cell in the upper left of the section that recorded a stress of around 150 kPa (see Fig. 5). Strain gauges in the 2nd shotcrete layer recorded a prevailing compressive strain with values in the range 400 to 550 $\mu\epsilon$ (see Fig. 6).

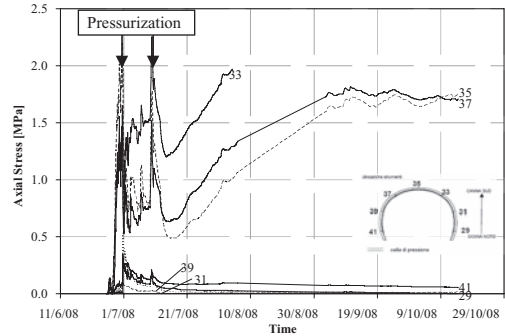


Figure 3. 1st shotcrete layer. Mean axial stress by pressure cells.

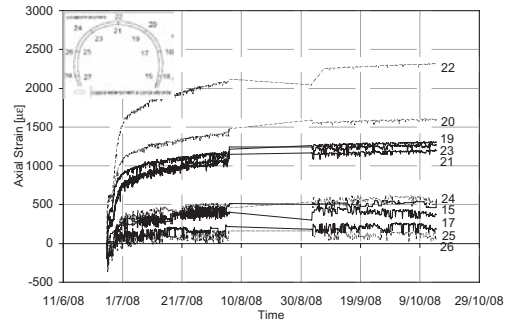


Figure 4. 1st shotcrete layer. Axial strain by strain gauges.

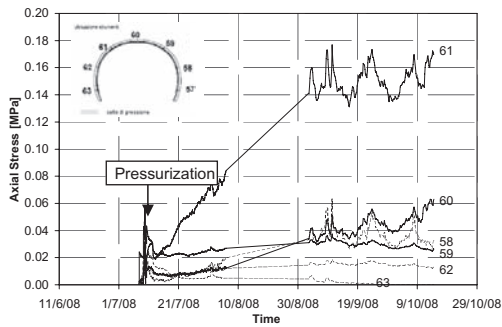


Figure 5. 2nd shotcrete layer. Mean axial stress by pressure cells.

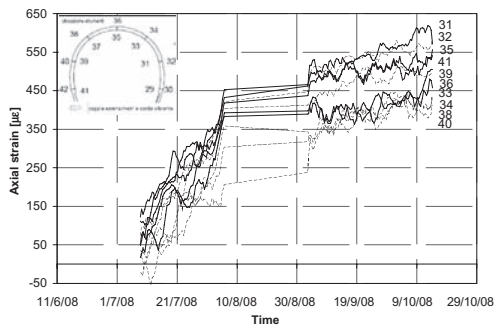


Figure 6. 2nd shotcrete layer. Axial strain by strain gauges.

As expected, values recorded in the 2nd layer were much lower than those of the 1st layer. In fact, the secondary lining was applied only after the whole excavation phase has been finished and then far from the excavation face. Despite this the growing stresses and strains observed over time (see Figures 5–6) confirmed the ability of the 2nd layer to carry part of the total load acting on the whole lining.

In Figures 7–9, 10, and 12, the distribution of stresses and strains in shotcrete layers and steel ribs are shown based on data readings at a time of approximately 100 days. The stress and strain patterns are close to the expected ones, since they show a prevalent compressive state and small bending moments. The only anomalous result is the diagram of average stresses for the pressure cells (Figures 7, 9) where stresses seem to fall to zero at the footings of the lining, especially in the case of the 1st layer. This is probably due to the arching effect around the pressure cell as previously described.

Observing the uniform strain distribution through the thickness of the secondary layer, the initial zero value, and the almost constant growth

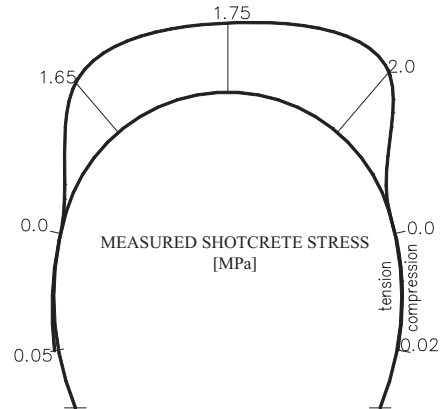


Figure 7. 1st shotcrete layer. Stress distribution by pressure cells.

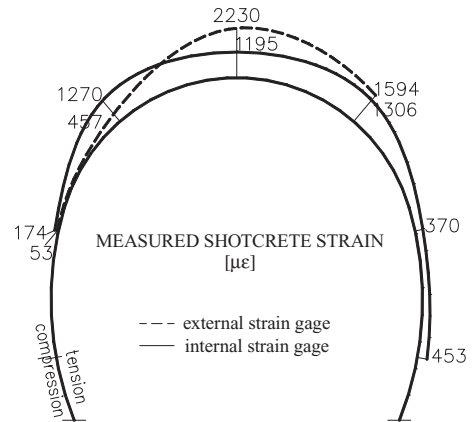


Figure 8. 1st shotcrete layer. Strain distribution by strain gauges.

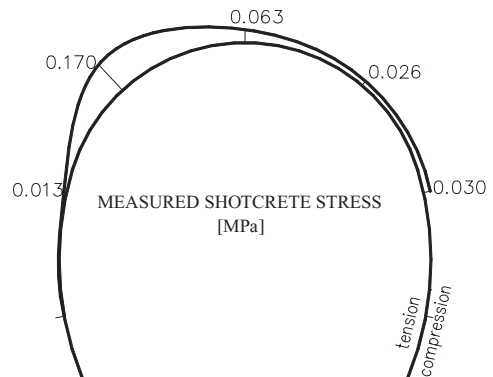


Figure 9. 2nd shotcrete layer. Stress distribution by pressure cells.

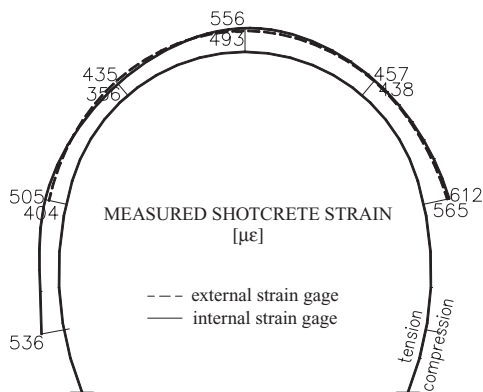


Figure 10. 2nd shotcrete layer. Strain distribution by strain gauges.

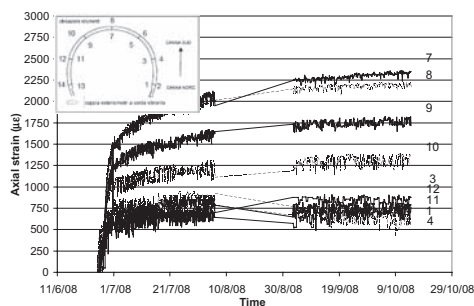


Figure 11. Steel ribs. Axial strain by strain gauges.

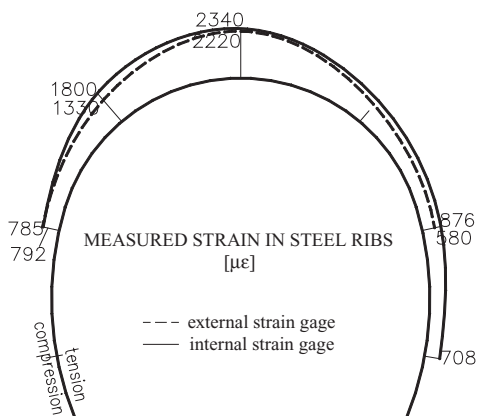


Figure 12. Steel ribs. Strain distribution by strain gauges.

over time (see Figures 6, 10), this appears to confirm that the two shotcrete layers act together as a composite structure highlighting the capability of the adopted sprayed-on waterproof membrane (performed with BASF Masterseal® 345)

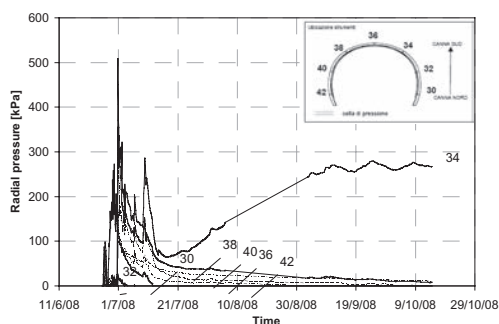


Figure 13. Ground pressure by tangential pressure cells.

of transferring shear stress through the shotcrete interface and thereby redistribute the load between the two layers.

Regarding the addition of the second phase lining, as a consequence of changing the cross-section, the creep and shrinkage of the two layers will affect each other, depending on the type of bonding between them. This is a significant issue in view of a further optimisation of the design of the section as a single composite system. A program of research has already been initiated in this direction.

5 LABORATORY TESTS RESULTS

In order to verify the suitability of using the standard curve for the purpose of modelling aging, creep and shrinkage behaviour in the shotcrete adopted for the experimental site, samples were taken and some laboratory tests were performed.

5.1 Elastic modulus determination

To estimate the elastic modulus of shotcrete at different ages, 20 cores (78.8 mm diameter, 114.0 mm height) of shotcrete with 4% accelerator dosage, were extracted from panels sprayed on site during a normal spraying cycle near the excavation face, and tested at 1, 2, 7, 14 and 28 days after production. For each testing age, one unconfined compression test (UCS) and three elastic modulus estimation tests were performed on different samples. The elastic modulus tests were performed at a standard temperature equal to $21 \pm 2^\circ\text{C}$ and $\text{RH} = 60 \pm 10\%$ (following UNI-EN-13412 Standard).

The three curves obtained by these tests (Figure 14) show similar results; moreover, the average curve is quite similar to the curve derived by applying the ASTM-C42 standard for concrete, properly scaled and adopting the mean strength at 28 days

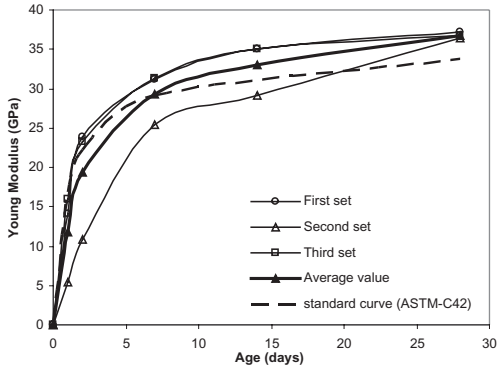


Figure 14. Elastic modulus experimental curves.

Table 2. Elastic modulus of shotcrete samples.

Sample	Dosage of accelerator (%)	Modulus of elasticity (GPa)
Mix-TQ (24h age)	0	11
Mix-4 (24h age)	4	16.1
Mix-8a (24h age)	8	7.8
Mix-8b (7d age)	8	21

obtained on the shotcrete samples by the uniaxial compressive tests.

5.2 Creep and shrinkage determination

To investigate the creep behaviour of the adopted shotcrete, various sustained load-controlled tests were performed on shotcrete samples containing the same admixtures except for a different dosage of accelerator. The shotcrete samples had a 225 cm² square base and were 62 cm high; samples were taken on site by spraying shotcrete directly in wood-boxes, varying the accelerator dosage in order to get samples with no accelerator (Mix-TQ), 4% (Mix-4), and 8% (Mix-8a) accelerator. For each accelerator percentage, a sustained load test was performed on one sample by loading it 24h after spraying, with a constant load equal to 3 MPa. Furthermore, an additional sample with 8% accelerator dosage was loaded 7 days after spraying (Mix-8b). In the meantime, additional samples with the same admixtures and dimensions as the loaded ones, were kept in the same ambient conditions in order to measure shrinkage. The creep tests were performed at a standard temperature equal to $20 \pm 3^\circ\text{C}$ and RH $50 \pm 5\%$.

In Table 2 the elastic modulus obtained at the first reading, 5 minutes after load application, is summarized. The differences shown between the samples at the same age are mainly due to the different dosage of accelerator (see Thomas, 2009): a low percentage

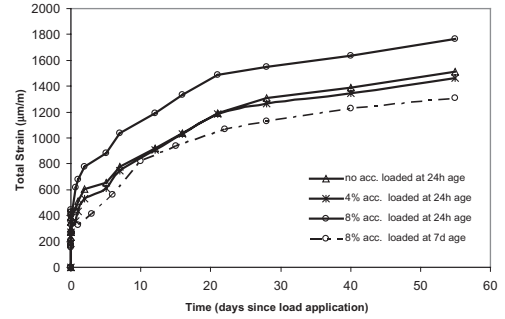


Figure 15. Total strain curves during sustained load tests.

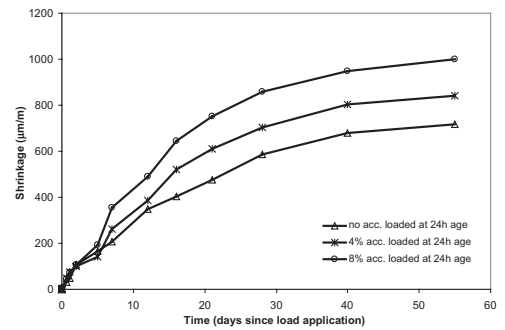


Figure 16. Shrinkage curves.

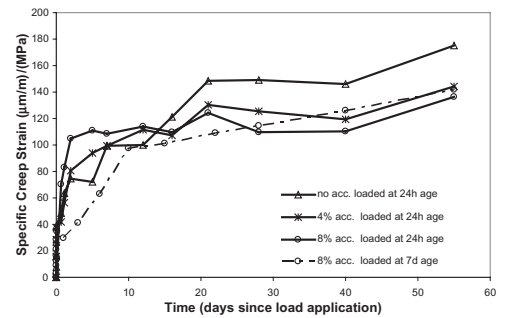


Figure 17. Specific creep strain curves.

of accelerator (4%) caused an increase of elastic modulus at 24h; a high percentage (8%) caused an increase in strength and stiffness at very early ages (during the first few hours) but a reduction later, compared to the original mix without accelerator.

In Figures 15–17 the main results of testing are summarized in terms of total strain (measured on loaded samples by removable extensometer), shrinkage (ie. total strain measured on unloaded samples by removable extensometer) and specific creep

strain (ie. total strain without initial elastic strain and shrinkage, divided by the applied stress).

It is clear in Figure 15 that the Mix 8a sample showed a worse overall behaviour over time as a consequence of both a lower elastic modulus at 24h (Table 2) and higher shrinkage during the first 20 days after production (Figure 16). Moreover, a comparison between the behaviour of Mix-8a and Mix-8b confirms the influence of the age at first loading on the instantaneous and delayed deformation.

Finally, the following conclusions can be drawn:

- The higher the dosage of accelerator, the higher the shrinkage strain. Compared to Mix-TQ, Mix8a showed a shrinkage strain 40% higher after less than 60 days,
- The creep behaviour is only lightly affected by the dosage of accelerator (Figure 16),
- Under the applied stress (3 MPa), more than half of the total strain (Figure 15) is due to shrinkage (Figure 17), confirming the necessity to take this phenomenon into account;
- The creep coefficient (ratio between creep strain at a certain time and initial elastic strain) obtained by test ranges between 1.2 and 2.7 at 60 days.

6 STRESS AND STRAIN IN COMPOSITE STEEL-CONCRETE CROSS-SECTION

Since the shotcrete exhibits the properties of creep and shrinkage, the two components forming the composite cross-section—the concrete matrix and steel reinforcement—tend to exhibit different strains. However, because of the bond between these two elements the difference in strain is restrained. Thus, the stresses in each section component change with time as creep and shrinkage develop.

6.1 Basic assumptions and sign conventions

This chapter is concerned with the calculation of the time-dependent stresses and the associated strains, in individual cross-section under the following assumptions and sign conventions: the cross-section is assumed to have one axis of symmetry and to be subjected to internal forces applied at a generic reference point “O” on axis; values of internal forces and time of their application are assumed to be known *a priori*; because of the low bending moments and predominant compressive axial force, no cracking is assumed; axial force is positive when tensile, while bending moment and curvature are positive when producing tension on the bottom (or inwards) fibers; creep and shrinkage change the distribution of stress and strain in the section but do not change the stress resultants

(values of the axial force and bending moment acting on the section, thus the structure is assumed to be statically determinate; tensile stress and the corresponding strain are positive, thus the value of shrinkage strain for concrete is generally a negative quantity; the Δ symbol represents an increment or decrement when positive or negative, respectively. In addition, perfect bond is assumed between concrete and steel, thus at any fibre the strain in the concrete and steel are equal; and plane cross-sections are assumed to remain plain after deformation, thus the strain distribution across the section is assumed to be linear according to the expression:

$$\varepsilon_c(y) = \varepsilon_s(y) = \varepsilon(y) = \varepsilon_o + \psi y \quad (2)$$

where y = distance from the reference point; $\varepsilon_c(y)$ = concrete strain at point in y ; $\varepsilon_s(y)$ = steel strain at point in y ; ε_o = strain at the reference point “O”; ψ = section curvature.

6.2 The modular ratio method

The simplest solution in order to get a first estimate of the stresses in the composite section is the classical, so called, “homogenisation technique”, or “modular ratio method”.

The method was devised in 1894 to avoid the problems involved in applying the theory of elasticity, developed in the 19th century for a material that was homogeneous, isotropic and of linear elastic behaviour such as iron, to a material that was non-homogeneous, anisotropic and elastic in anomalous manner, such as reinforced concrete. This rendering of iron so that it is equivalent to cement is achieved by a “homogenisation coefficient” n , defined by the ratio between the elastic modulus of steel and concrete:

$$n = E_s/E_c \quad (3)$$

The “ideal homogeneous section” and the n coefficient were used in national codes and in engineering practice until recently, at least in Italy, for simplified linear-elastic analysis of reinforced concrete and mixed steel-concrete members:

- a. The Old Italian Technical Standards for Construction (1996) sets a default value of $n = 15$ for the homogenisation coefficient and gives for elastic modulus of concrete at $t = 28$ days:

$$E_c = 5700 \cdot \sqrt{R_{ck}} \text{ (MPa)} \quad (4)$$

where R_{ck} is the characteristic compressive strength measured using cubic specimens.

- b. The Italian Instructions for Steel-concrete Composite Structures CNR 10016 (2000)

gives a different formula for secant modulus at $t = 28$ days

$$E_{c0} = 9500 \cdot \sqrt[3]{(f_{ck} + 8)} \text{ (MPa)} \quad (5)$$

and suggests use of the value of E_{c0} for short-term loading and $E_c = E_{c0}/3$ for long-term loading.

- c. Eurocode 4 (2004) gives for linear elastic analysis of composite sections a value of the homogenization coefficient related to the creep coefficient φ_t and the type of loading, by means of the parameter ψ_L ($\psi_L = 1.1$ for permanent loads or $\psi_L = 0.55$ for shrinkage)

$$n_L = n_0 \cdot (1 + \psi_L \cdot \varphi_t) \quad (6)$$

where n_0 is the modular ratio for short-term loading, relative to the secant modulus of elasticity of the concrete at 28 days E_{cm} computed as

$$E_{cm} = 22000 \cdot (f_{cm}/10)^{0.3} \text{ (MPa)} \quad (7)$$

- d. The New Italian Technical Standards for Construction (2008) refers to Eurocode 4 for the linear elastic analysis of steel-concrete composite sections, but suggests to use a default value $E_c = 0.5 \times E_{cm}$ for the modulus of elasticity of the concrete for long-term loading.

The design of temporary supports made by steel ribs and shotcrete, in order to get a rough estimate of the stresses, often assumes that such temporary lining simply behaves as a reinforced concrete section. Unfortunately, such an approach often leads to completely misleading results since shotcrete is loaded at early ages and shows a different creep and shrinkage behaviour compared to cast in situ concrete (Kusterle 1992, Thomas 2009); hence, a different secant elastic modulus should be used.

In fact, adopting the above mentioned approaches with a rectangular concrete section measuring 100×20 cm ($A_c = 2000$ cm², $f_{ck} = 28$ MPa), reinforced with a pair of double-T steel ribs IPN180 each 1.2 m ($A_s = 55.8/1.2 = 46.5$ cm², $E_s = 205$ GPa) and loaded with a permanent axial force $N = 1490$ kN, it results in a considerably different stresses compared to the experimental data, as shown in Table 3.

6.3 Creep and shrinkage of concrete

Because the stress and strain in a reinforced concrete structure are subjected to change for a long period of time during which creep and shrinkage of concrete develops gradually, the analysis of the time-dependent stresses and deformations must incorporate time-dependent functions for strain or stress in the materials involved. Many international

Table 3. Elastic sectional analysis with the modular ratio method: stresses due to long-term loading.

Case	E_{c28} (GPa)	E_c (GPa)	$n = E_s/E_c$ (-)	$A_c + nA_s$ (cm ²)	σ_c (MPa)	σ_s (MPa)
a	33.7	—	15	2700	5.5	83
b	31.5	10.5	19.6	2900	5.1	101
c	32.3	—	28	3300	4.5	126
d	32.4	16.2	13	2600	5.7	75

Codes—such as MC-90, ACI Committee and British Standards BS 8110—give expressions and graphs for time effects in concrete (development of strength and modulus of elasticity with time, creep coefficient, shrinkage), including in the equations the important parameters that affect stresses or strains.

In this paper we refer, as a first approximation, to the linear visco-elastic model, valid for ordinary structural concrete ($12 \text{ MPa} < f_{ck} < 80 \text{ MPa}$) subjected to a compressive stress $\sigma_c < 0.4 f_{cm}$ and exposed to mean relative humidity in the range of 40 to 100% at mean temperatures from 5°C to 30°C, but assuming modulus of elasticity, creep and shrinkage values from experimental data obtained for structural shotcrete. The following basic equations and principles are assumed to be valid.

6.3.1 Strain components

The total strain is composed as follows:

$$\begin{aligned} \varepsilon_c(t) &= \varepsilon_{c\sigma}(t) + \varepsilon_{cs}(t) \\ \varepsilon_{c\sigma}(t) &= \varepsilon_{ci}(t_0) + \varepsilon_{cc}(t) \end{aligned} \quad (8)$$

where $\varepsilon_{c\sigma}(t)$ = stress dependent strain; $\varepsilon_{cs}(t)$ = stress independent strain (shrinkage); $\varepsilon_{ci}(t_0)$ = instantaneous strain at loading time t_0 ; $\varepsilon_{cc}(t)$ = creep strain at time t .

The instantaneous strain at loading time t_0 is

$$\varepsilon_{c\sigma}(t_0) = \varepsilon_{ci}(t_0) = \sigma_c(t_0)/E_c(t_0) \quad (9)$$

where $\sigma_c(t_0)$ = concrete stress; $E_c(t_0)$ = modulus of elasticity at age t_0 .

The stress-dependent strain at time t is

$$\varepsilon_{c\sigma}(t) = \varepsilon_{ci}(t_0) + \varepsilon_{cc}(t) = [\sigma_c(t_0)/E_c(t_0)] \cdot [1 + \varphi(t, t_0)] \quad (10)$$

where $\varphi(t, t_0)$ is a dimensional coefficient that is a function of the age at loading, t_0 and the age t for which the strain is calculated, representing the ratio of creep to the instantaneous strain:

$$\varphi(t, t_0) = \varepsilon_{cc}(t)/\varepsilon_{ci}(t_0) \quad (11)$$

As regards shrinkage strain, since the stresses induced by shrinkage are generally reduced by the effects of relaxation of concrete, in the stress analysis both the effects of these two phenomena must be considered simultaneously. For this purpose, the amount of free shrinkage and an expression for its variation in time are needed, with a remark: shrinkage starts to develop at time t_s when moist curing stops, not necessarily the time t_0 corresponding to application of loads. In our case, since the application of the load is very early, it was chosen for convenience to simplify the calculation and to match t_s with t_0 .

6.3.2 Creep superposition principle

Eqn. (10) implies the assumption that the total stress-dependent strain, instantaneous plus creep, is proportional to the applied stress. This linear relationship allows the principle of superposition of the strains due to stress increments or decrements and due to shrinkage, leading to the following integral law:

$$\varepsilon_c(t) = \sigma_c(t_0) \frac{1 + \varphi(t, t_0)}{E_c(t_0)} + \int_0^{\Delta\sigma_c(t)} \frac{1 + \varphi(t, \tau)}{E_c(\tau)} d\sigma_c(\tau) + \varepsilon_{cs}(t, t_0) \quad (12)$$

6.3.3 Ageing coefficient and algebraic formulation

An increment in the applied stress $\Delta\sigma_c$, gradually introduced during the period t_0 to t , produces lower levels of creep than the same increase $\Delta\sigma_c$ if applied in t_0 and sustained during the period $(t - t_0)$. The effect of a gradual stress increment from zero at t_0 to a value $\Delta\sigma_c$ at time t , can be treated the same way as the stress $\Delta\sigma_c$ applied instantly at t_0 , through the use of a reduced creep coefficient $\chi\varphi(t, t_0)$, where $\chi(t, t_0)$ is an aging coefficient whose value should be less than 1. Values of χ are theoretically in between the extreme limits of ~ 0.5 and 1, but in practical cases vary between 0.6 and 0.9.

Based on the algebraic formulation by Trost (1967), the introduction of the ageing coefficient χ allows an important simplification of (12), which can be rewritten as follows:

$$\varepsilon_c(t) = \sigma_c(t_0) \frac{1 + \varphi(t, t_0)}{E_c(t_0)} + \Delta\sigma_c(t) \frac{1 + \chi\varphi(t, t_0)}{E_c(t_0)} + \varepsilon_{cs}(t, t_0) \quad (13)$$

The ageing coefficient $\chi(t, t_0)$ is related to the relaxation function $r(t, t_0)$ by the equation

$$\chi(t, t_0) = \frac{1}{1 - r(t, t_0)/E_c(t_0)} - \frac{1}{\varphi(t, t_0)} \quad (14)$$

so the calculation of $\chi(t, t_0)$ requires the step-by-step numerical integration of the relaxation function for concrete $r(t, t_0)$, which requires an analytical knowledge of the creep function or at least its numerical value for each interval (t_i, t_j) , with t_i and t_j ranging on all time-steps in which the (t_0, t) interval has been subdivided. Having analytical formulations for the creep law, values of $r(t, t_0)$ and $\chi(t, t_0)$ are usually reported in tables and graphs annexed to the standard codes thereby avoiding troublesome calculations.

Trost's suggestion was to use $\chi = 0.8$ as a constant value for the long-term case ($t = \infty$), regardless of the value of t_0 . Nevertheless, with modern formulations of the creep function (ie. CEB-FIP MC90) and very early time t_0 , χ assumes values less than 0.8 and closer to 0.5. In any case, with the *a priori* assumption of the ageing coefficient the relaxation function is no longer required.

6.4 Age-adjusted elastic modulus method

The terms in Equation (13) represent the three components of strain in concrete at time t due to:

- stress $\sigma_c(t_0)$ applied at time t_0 and sustained during the interval $(t - t_0)$,
- increase in stress with value gradually rising from zero at t_0 to $\Delta\sigma_c(t)$ at time t ,
- shrinkage measured in the period $(t - t_0)$.

Through the introduction of an age-adjusted elastic modulus $E'_c(t, t_0)$, Equation (13) can be rewritten as follows

$$\varepsilon_c(t) = \sigma_c(t_0) \frac{1 + \varphi(t, t_0)}{E_c(t_0)} + \frac{\Delta\sigma_c(t)}{E'_c(t, t_0)} + \varepsilon_{cs}(t, t_0) \quad (15)$$

$$E'_c(t, t_0) = \frac{E_c(t_0)}{1 + \chi\varphi(t, t_0)} \quad (16)$$

so that the increase in strain produced in the interval $(t - t_0)$ by the increase in stress $\Delta\sigma_c(t)$ is

$$\Delta\varepsilon_c(t) = \Delta\sigma_c(t)/E'_c(t, t_0) \quad (17)$$

Likewise for the modular ratio method, in the Age-Adjusted Elastic Modulus Method (AAEMM), the effective section is replaced by a “transformed” section given by the sum of the concrete area and the steel area multiplied by n , where

$$n(t_0) = E_s/E_c(t_0) \quad \text{and} \quad \bar{n}(t, t_0) = E_s/E'_c(t, t_0) \quad (18)$$

6.5 Elastic analysis

As a consequence of Eqs. (2), (8) and (9), the initial elastic stress at $t = t_0$ is

$$\sigma(t_0) = [\varepsilon_o(t_0) + \psi(t_0)y - \varepsilon_{cs}(t_0)] \cdot E(t_0) \quad (19)$$

where y = distance from the reference point (positive when the point considered is below the reference point); $E(t_0)$ = instantaneous elastic modulus at time t_0 of the fibre at distance y ; $\sigma(t_0)$ = instantaneous elastic stress at time t_0 of the fibre at distance y ; $\varepsilon_o(t_0)$ = strain at the reference point "O" in t_0 ; $\psi(t_0)$ = section curvature in t_0 ; $\varepsilon_{cs}(t_0)$ = shrinkage at $t = t_0$ of the concrete fibre at distance y .

Assuming the shrinkage strain at time t_0 is equal to zero and integrating over the section for the stresses obtained by (19), the consequent internal forces calculated with respect to the axis that passes at the reference point O are obtained as

$$\begin{aligned} \begin{Bmatrix} \varepsilon_o \\ \psi \end{Bmatrix} &= \frac{1}{E'} \begin{bmatrix} A' & B' \\ B' & I' \end{bmatrix}^{-1} \begin{Bmatrix} N \\ M \end{Bmatrix} \\ &= \frac{1}{E'(A'I' - B'^2)} \begin{bmatrix} I' & -B' \\ -B' & A' \end{bmatrix} \begin{Bmatrix} N \\ M \end{Bmatrix} \end{aligned} \quad (20)$$

where $A' = n_c A_c + n_s A_s$; $B' = n_c B_c + n_s B_s$; $I' = n_c I_c + n_s I_s$; $n_c = E_c/E'$; $n_s = E_s/E'$; E' = reference elastic modulus (usually equal to E_c); E_c and E_s = concrete and steel elastic modulus respectively; A_c and A_s = respectively concrete and steel area; B_c and B_s = respectively static moment of the concrete and steel (with respect to the reference axis passing at point O); I_c and I_s = respectively moment of inertia of the concrete and steel (with respect to the reference axis).

Equations (20) and (19) are the basic equations for computation of elastic stress and strain on the section as a consequence of the instantaneous load application at time t_0 .

6.6 Time-dependent analysis

There are two possible calculation approaches: the first, adopting the algebraic formulation (13), is manually feasible, but requires the knowledge of the aging coefficient χ ; the second, based on numerical integration of the law (12), is an iterative procedure well suited to use with a computer. In the following the first approach was adopted considering the unavailability of an analytical constitutive law for shotcrete and the availability of experimental data for only the two cases of an initial loading time $t_1 = 1$ day and $t_2 = 7$ days. The computation technique is detailed in Ghali, Favre & Elbardy (2002) and is briefly described below.

Assume that initial stress and strain (at time t_0) are known; creep and shrinkage phenomena then cause a stress redistribution through the composite section. The change in strain during the period t_0 to t is defined by the increments $\Delta\varepsilon_o$ and $\Delta\psi$ in the axial strain and curvature. The change in concrete strain due to creep and shrinkage is first artificially restrained by an application of an axial force ΔN and a bending moment ΔM .

Subsequently these restraining forces are removed by the application of equal and opposite forces on the composite section, resulting in the following changes in axial strain and curvature:

$$\begin{aligned} \begin{Bmatrix} \Delta\varepsilon_o \\ \Delta\psi \end{Bmatrix} &= \frac{1}{E'_c(A'I' - B'^2)} \begin{bmatrix} I' & -B' \\ -B' & A' \end{bmatrix} \begin{Bmatrix} -\Delta N \\ -\Delta M \end{Bmatrix} \\ \begin{Bmatrix} \Delta N \\ \Delta M \end{Bmatrix} &= \begin{Bmatrix} \Delta N \\ \Delta M \end{Bmatrix}_{\text{creep}} + \begin{Bmatrix} \Delta N \\ \Delta M \end{Bmatrix}_{\text{shrinkage}} \end{aligned} \quad (21)$$

where A' , B' , I' , are, respectively, the area of the age-adjusted transformed section and its first and second moment about an axis through the reference point O; $E'_c = E'_c(t, t_0)$ is the age-adjusted effective modulus defined in Eq. (16).

The restraining forces can be calculated by the following formula, derived from Eq. (20):

$$\begin{Bmatrix} \Delta N \\ \Delta M \end{Bmatrix}_{\text{creep}} = -E'_c \cdot \varphi \cdot \begin{bmatrix} A_c & B_c \\ B_c & I_c \end{bmatrix} \begin{Bmatrix} \varepsilon_o(t_0) \\ \psi(t_0) \end{Bmatrix} \quad (22)$$

$$\begin{Bmatrix} \Delta N \\ \Delta M \end{Bmatrix}_{\text{shrinkage}} = -E'_c \cdot \varepsilon_{cs} \cdot \begin{Bmatrix} A_c \\ B_c \end{Bmatrix} \quad (23)$$

where A_c , B_c , I_c , are, respectively, the area of the concrete section and its first and second moment about an axis through the reference point O; $E'_c = E'_c(t, t_0)$ is the age-adjusted effective modulus defined in Eq. (16); $\varphi = \varphi(t, t_0)$ is the creep coefficient. It is to be noted that age-adjusted modulus of elasticity is used in Eqs. (21), (22) and (23) because the forces ΔN are gradually developed between t_0 and t .

The stress increment that develops in the concrete during the period $(t - t_0)$ is as follows:

$$\begin{aligned} \Delta\sigma_c &= \sigma_{\text{restrained}} + E'_c(t, t_0) \cdot (\Delta\varepsilon_o + y\Delta\psi) \\ \sigma_{\text{restrained}} &= -E'_c(t, t_0) \cdot [\varphi(t, t_0) \cdot \varepsilon_c(t_0) + \varepsilon_{cs}] \end{aligned} \quad (24)$$

where $\sigma_{\text{restrained}}$ is the stress in concrete required to prevent creep and shrinkage at any fiber. Substituting $\sigma_{\text{restrained}}$ into the $\Delta\sigma_c$ expression, it follows that:

$$\Delta\sigma_c = E'_c(t, t_0) \cdot [(\Delta\varepsilon_o + y\Delta\psi) - \varphi(t, t_0) \cdot \varepsilon_c(t_0) - \varepsilon_{cs}] \quad (25)$$

Table 4. Time scales.

Mix	t_1^* (days)	t_0^{**} (days)	t_1^{***} (days)	t_2^{***} (days)
Mix-TQ	1	1	29	56
Mix-4	1	1	29	56
Mix-8a	1	1	29	56
Mix-8b	1	7	29	56

* t_s = age of concrete at the beginning of shrinkage

** t_0 = age of concrete at loading

*** t_1, t_2 = age of concrete at the moment considered

Table 5. Measured shrinkage and specific creep coefficient.

Mix	$\varepsilon_{sh}(t_1, t_s)$ ($\times 10^{-6}$)	$\varphi(t_1, t_0)$ (–)	$\varepsilon_{sh}(t_2, t_s)$ ($\times 10^{-6}$)	$\varphi(t_2, t_0)$ (–)
Mix-TQ	–568	1.65	–712	1.95
Mix-4	–705	2.02	–842	2.33
Mix-8a	–860	0.91	–1000	1.14
Mix-8b	–860	2.08	–1000	2.51

In the mean time, the stress increment that develops in the steel during the period $(t - t_0)$ is:

$$\Delta\sigma_s = E_s \cdot (\Delta\varepsilon_o + y\Delta\psi) \quad (26)$$

6.7 Case study

The time-dependent behaviour of the trial tunnel lining cross-section was analysed through the previous derivations, assuming the time-scale described in Table 4 and the simplified load condition of a permanent uniform axial force equal to 1490 kN/m, as estimated in Paragraph 3. The mechanical properties of shotcrete (elastic modulus, creep coefficient, shrinkage) were taken to correspond to those obtained through the laboratory tests performed on samples cast in situ (see Table 5).

Because the second layer (thinner and later applied compared to the first layer) monitored in the experimental testing gallery showed a low stress level, it was decided to ignore the effect of the joining provided by the waterproofing membrane between the two layers and to consider the second layer only as a non-structural protective coat of the membrane. So in the following we deal with the calculation of the stress redistribution between the ribs made with a double steel IPN180 profile placed every 1.2 metres, and the first permanent layer having a thickness of 20 cm.

A comparison with the creep parameters by CEB-FIP Model Code 1990 (see Table 6), valid for ordinary concretes, although with the same f_{ck} value,

Table 6. MC-90 shrinkage and creep parameters ($t_0 = 1$ day).

	$\varepsilon_{sh}(t_1, t_s)$ ($\times 10^{-6}$)	$\varphi(t_1, t_0)$ (–)	$\chi(t_1, t_0)$ (–)	$\varepsilon_{sh}(t_2, t_s)$ ($\times 10^{-6}$)	$\varphi(t_2, t_0)$ (–)	$\chi(t_2, t_0)$ (–)
MC-90	–41	0.88	0.54	–58	1.07	0.52

Table 7. Elastic stress and strain at loading.

Mix	$E_c(t_0)$ (GPa)	$n(t_0)$ (–)	$\varepsilon_c(t_0)$ ($\times 10^{-6}$)	$\sigma_c(t_0)$ (MPa)	$\sigma_s(t_0)$ (MPa)
Mix-TQ	11	18.6	–476	–5.2	–98
Mix-4	16.1	12.7	–359	–5.8	–74
Mix-8a	7.8	26.3	–597	–4.7	–122
Mix-8b	21	9.8	–291	–6.1	–60
MC-90	19	10.8	–316	–6.0	–65

was also carried out to investigate the consequence of the unavailability of experimental data and the absence of an analytical constitutive law for shotcrete provided by codes. Note that the symbol φ_{CEB} was used for the creep coefficient employed in MC-90, being equal to the following relationship:

$$\varphi = \frac{E_c(t_0)}{E_c(28)} \cdot \varphi_{CEB} = \frac{19}{33} \cdot \varphi_{CEB} = 0.58 \cdot \varphi_{CEB} \quad (27)$$

where $E_c(28) = 33$ GPa is the modulus of elasticity at age 28 days and $E_c(t_0) = 19$ GPa is the modulus of elasticity at age $t_0 = 1$ day. Other parameters affecting creep in the MC-90 formulation are:

$$h_0 = 2A_c / u; \text{RH}; \alpha \quad (28)$$

where: h_0 is the notional size (in mm); A_c and u are the area and perimeter in contact with the atmosphere of the cross-section; RH (%) is the relative humidity; and α is a parameter depending on the type of cement. In the calculations values of $f_{ck} = 28$ MPa, $h_0 = 400$ mm, RH = 50% (dry conditions) and $\alpha = 0$ (normal or rapid hardening cements) were used. The results are summarized in the following.

Through comparison of Tables 5 and 6 it is evident that the MC-90 law for normal concrete is inadequate to fit the real creep behaviour of shotcrete and especially underestimates shrinkage. The results highlighted in Table 10 are better than those obtained by the more simple “modular ratio method”, adopting the homogenisation coefficient “ n ”, but they differ substantially from those derived through the use of the experimental data and laboratory tests. The laboratory results have

Table 8. Total stress and strain at time $t - \chi = 0.6$.

Mix	$\varepsilon_o(t) (\times 10^{-6})$		$\sigma_c(t) \text{ (MPa)}$		$\sigma_s(t) \text{ (MPa)}$	
	t_1	t_2	t_1	t_2	t_1	t_2
Mix-TQ	-1202	-1321	-1.8	-1.2	-246	-271
Mix-4	-1224	-1342	-1.7	-1.1	-251	-275
Mix-8a	-1318	-1425	-1.2	-0.7	-270	-292
Mix-8b	-1134	-1250	-2.1	-1.6	-233	-256

Table 9. Total stress and strain at time $t - \chi = 0.9$.

Mix	$\varepsilon_o(t) (\times 10^{-6})$		$\sigma_c(t) \text{ (MPa)}$		$\sigma_s(t) \text{ (MPa)}$	
	t_1	t_2	t_1	t_2	t_1	t_2
Mix-TQ	-1127	-1223	-2.1	-1.7	-231	-251
Mix-4	-1140	-1236	-2.1	-1.6	-234	-253
Mix-8a	-1261	-1348	-1.5	-1.1	-259	-276
Mix-8b	-1065	-1158	-2.5	-2.0	-218	-238

Table 10. MC-90 total stress and strain at time t .

Mix	$\varepsilon_o(t) (\times 10^{-6})$		$\sigma_c(t) \text{ (MPa)}$		$\sigma_s(t) \text{ (MPa)}$	
	t_1	t_2	t_1	t_2	t_1	t_2
MC-90	-548	-600	-4.9	-4.6	-112	-123

Table 11. Stress in the steel ribs (MPa).

Mix	Elastic $t_0 = 1 \text{ day}$	χ	t_1 29 days	t_2 56 days
Measured	-64		-240	-255
Computed				
Mix-TQ	-98	0.6	-246	-271
		0.9	-231	-251
Mix-4	-74	0.6	-251	-275
		0.9	-234	-253
Mix-8a	-122	0.6	-270	-292
		0.9	-259	-276

therefore proved to be instructive for measuring the parameters properly and the monitoring system useful to check the results of the calculation method. This is also essential to a proper design approach and dimensioning, thus resulting in a greater economy and safety.

By the results summarized in Tables 8 and 9, it is clear that the effect of the ageing factor is negligible on the computed stress (less than 10%). Furthermore, the influence of the accelerator dosage

Table 12. Strain in the 1st shotcrete layer ($\mu\epsilon$).

Mix	Elastic $t_0 = 1 \text{ day}$	χ	t_1 29 days	t_2 56 days
Measured	-392		-1160	-1240
Computed				
Mix-TQ	-476	0.6	-1202	-1321
		0.9	-1127	-1223
Mix-4	-359	0.6	-1224	-1342
		0.9	-1140	-1236
Mix-8a	-597	0.6	-1318	-1425
		0.9	-1261	-1348

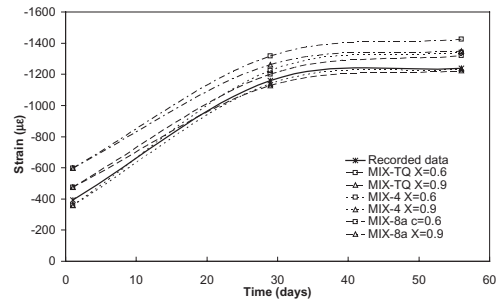


Figure 18. Comparison between computed and measured average strain in the 1st shotcrete layer.

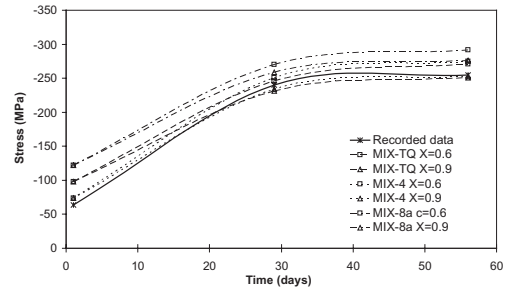


Figure 19. Comparison between computed and measured average stress in the steel ribs.

on the result diminishes with time as creep and shrinkage deformations develop.

In Tables 11 and 12 the strains and stresses computed when adopting parameters derived from samples loaded at 24 hours are compared to the experimental data obtained by strain gauges. As shown by Figures 18–19, computed strains and stresses are very similar to the measured values especially for the case of shotcrete sample with 4% accelerator dosage.

It can also be observed that the values of the ratio between stresses in the steel and in the shotcrete, as shown in Tables 8–9, vary between 110 and

225 at time $t_1 = 29$ days and between 120 and 410 at time $t_2 = 56$ days, values that are much higher than those obtained by simple elastic calculation, such as the “modular ratio method”, using homogenisation coefficients which vary between 15 and 30.

7 CONCLUSIONS

In order to use shotcrete as a permanent lining in service by-pass tunnels, a monitoring system was installed in a trial tunnel. Rigorous laboratory tests were also performed in order to investigate the actual behaviour of the adopted shotcrete mix design.

The experimental data recorded by monitoring instruments show that stresses in the lining are quite different from values estimated by the simple “modular ratio method”. This confirmed the inadequacy of such simplified design method in simulating the time-dependent behaviour of shotcrete linings reinforced with steel ribs.

Using the parameters obtained by the laboratory tests, an improved calculation approach for estimating stresses in the shotcrete layers and in the steel ribs was developed in which aging, shrinkage and creep behaviour of shotcrete are considered. By means of this approach, stress values similar to the measured ones were obtained.

The comparison with the time-effects parameters in the CEB-FIP Model Code 1990 for ordinary concretes showed their inadequacy to modelling shotcrete behaviour, especially with regard to shrinkage.

The results of the investigation and measurements confirmed the necessity to adopt an improved design approach to the modelling of long-term behaviour, as proposed, that considers the complex behaviour of shotcrete in particular at early ages. The current evidence, indicating that the increment of stresses in the steel ribs may be mainly due to delayed effects such as shrinkage and creep rather than rock overload is a completely new interpretation of these phenomena.

Furthermore, the similarity between the results of the improved approach and the monitoring data confirmed the reliability of the installed monitoring system and, consequently, its adequacy for future applications. Regarding the monitoring data systems, when starting from the strains in the tunnel lining obtained by means of strain gauges, it is mandatory to consider the correct shotcrete time-dependent behaviour. The correct value of the initial elastic strain at the very initiation of loading must be used. This is a function of the elastic modulus, and in this regard use of the secant modulus at 28 days has proved to be unrealistic in case of shotcrete tunnel lining. The creep

and shrinkage components of strains in the overall measured deformation must be considered, and these have proved to be particularly significant in the case of shotcrete linings. The same also applies for the interpretation of monitoring data gathered by instruments that are embedded in plain or reinforced concrete linings (or welded to its reinforcement bars), since it is also loaded at early ages.

Failure to properly consider all the time-dependent effects might lead to significantly overestimated loads in the lining when performing back-calculation of the stresses from strains. Such incorrect load values can become so significant that useless remedial works may be implemented with consequent loss of time and large increase of costs.

In the current case study, it was found that the steel carries 80% of the ground load, yet has an area of 5% of the concrete, which means that if you need a stiffer support, you should put in more steel, not more shotcrete.

Another result of the case study was that the ageing coefficient χ does not appear to be a sensitive number. However, more tests are needed with different shotcrete mixes and different times at loading t_0 . Only in this way it will be possible to have a broader view of the effect of different values of elastic modulus and creep curves and to back calculate the relaxation curve and thus also calculate the ageing coefficient χ . On the other hand, for the particular type of application, it appears that the time of load is the dominant factor compared to those related to the aging of the material. Perhaps for this reason the magnitude of the aging coefficient appears to have little effect but this can not become a firm assertion in any other case.

Concerning the importance of shrinkage relative to creep it was found that the shrinkage of sprayed concrete is much greater (about 10 times) than that of standard cast concrete. For this reason it has even greater importance compared to the deformation due to creep, especially within concrete elements that are loaded with a very low level of initial stress. In permanent structures, it will certainly be useful to work towards reducing the shrinkage: more curing, many thin coats rather than one thick layer made in a single pass, more compaction in spraying, and so on.

The case study has dealt with the calculation of the stress redistribution between steel ribs and shotcrete of the first permanent layer, subjected to a constant load fully applied in a single step at $t_0 = 1$ day. The simplifying assumption that all of the ground load comes on at first day onto the first layer was an approximation necessary for the initial understanding of the stress redistribution phenomena and it was adopted in the light of the reasons detailed in the paper. The next goal is to

develop a similar method of calculation which can consider a multiple-step load transfer over time, so that the relaxation curves can be followed.

At a later date another objective will be to evaluate the behaviour of a section composed by two shotcrete layers with an interposed sprayed-on membrane, working as a single shell with a yielding embedded constraint. The mechanical behaviour of the membrane depends on several parameters such as stiffness, thickness, relative position within the whole section, if near or far to the neutral axis, timing of the membrane spraying related to the first and second shotcrete layers building, creep and relaxation of the polymeric matrix compared to the creep and relaxation of the shotcrete. This needs a wider and expensive research program, which is currently under development.

An additional laboratory test program is under way to better investigate the interaction between steel ribs and shotcrete at early ages as a consequence of shrinkage, and support the improved approach here proposed.

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REFERENCES

- Bazant, Z.P. 1972. Prediction of Concrete Creep Effects using Age Adjusted Effective Modulus Method, *ACI Journal*, N. 69.
- Comité Euro-International du Béton (CEB), 1991. CEB-FIP Model Code 1990. *Final Draft*, CEB Bulletin d'Information, N. 203, July 1991, Lausanne.
- European Committee for Standardisation (CEN), 2004. European Standard EN 1994-1-1. *Eurocode 4: Design of composite steel and concrete structures. Part 1-1: General rules and rules for buildings*, CEN, Brussels.
- Ghali, A., Favre, R. & Elbadry, M. 2002. *Concrete Structures. Stresses and Deformations*, 3rd ed., Spon Press, London & New York.
- Golser, J. & Kienberger, G. 1997. *Permanente Tunnelauskleidung in Spritzbeton—Beanspruchung und Sicherheitsfragen*, Felsbau, 416–21.
- Italian Ministry of Infrastructure, 2008. *New Technical Standards for Construction* (in Italian), GURI (Official Journal of the Italian Republic), Vol. 29, 4th February 2008, Rome.
- Italian Ministry of Public Works, 1996. *Technical standards for the calculation, execution and validation of reinforced concrete structures, normal and pre-stressed, and for metal structures* (in Italian), GURI (Official Journal of the Italian Republic), Vol. 29, 5th February 1996, Rome.
- Italian National Research Council (CNR), 2000. Technical instructions 10016. *Steel-concrete Composite Structures. Instructions for use in buildings* (in Italian), CNR Official Journal, Vol. XXXIV, N. 194, September 2000, Rome.
- Kursterle, W. 1992. *Qualitätsverbesserungen beim Spritzbeton durch technologische Massnahmen, durch den Einsatz neuer Materialien und aus Grund der Erfassung von spritzbetoneigenschaften*. Habilitationsschrift, University of Innsbruck.
- Mola, F. 1985. Long term analysis of steel-concrete beams (in Italian), *Studies & Researches*, Vol. 11, Italcementi, Bergamo.
- Thomas, A.H. 2009. *Sprayed concrete lined tunnels*. Taylor & Francis, London.
- Trost, H. 1967. Auswirkungen des Superpositionprinzips auf Kriech und Relaxationprobleme bei Beton und Spannbeton, *Beton und Stahlbetonbau*, H.10.

Economical mix design enhancements for FRS

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ABSTRACT: As part of a program of research to develop high performance economical shotcrete for use in ground support within underground mines, several trials were undertaken by Newcrest Mining Ltd and TSE P/L to investigate the influence of admixtures and silica additives on the wet properties and early-age strength characteristics of Fibre Reinforced Shotcrete (FRS). The issues examined in these trials were: the influence of a hydration stabilizer and an Air Entraining Agent (AEA) on slump and setting characteristics, the influence of synthetic micro-fibres on spraying characteristics, and the influence of silica fume substitutes on shotcrete performance. The objectives were: to identify an economical means of maintaining slump for sustained periods of time, of achieving improved spraying and adhesion characteristics, and of sustaining or increasing the rate of early-age strength gain following spraying.

1 INTRODUCTION

Newcrest Mining Ltd (NML) is Australia's largest gold producer and a global top 10 gold mining company by production, reserves, and market capitalisation. Newcrest Mining Ltd currently excavates approximately 40–50,000 metres of horizontal development annually and uses 60–70,000 m³ of shotcrete annually across its underground mines and development projects (Figure 1). The cost of shotcrete used annually in NML mines is currently between A\$35–40 million and this is forecast to continue for the next three years.

Based on the significant volume of shotcrete and associated costs, NML has allocated funding to research areas that will result in improvements in shotcrete design, quality, and costs. The current paper describes the results of recent investigations into shotcrete mix design improvement (Bernard, 2008a, 2008b, 2008c). A tripartite team with representatives from NML (customer), TSE Pty Ltd (research/testing) and Stratacrete Pty Ltd (experienced applicator) was established in May 2008 to coordinate the research effort. A staged approach to research has been implemented to date:

- Stage 1: Laboratory tests of admixtures such as super-plasticisers, silica fume replacements, and air entraining agents.
- Stage 2: Underground trials of selected admixtures with site mix designs.
- Stage 3: Implementing better mix specifications into production and improving underground application.

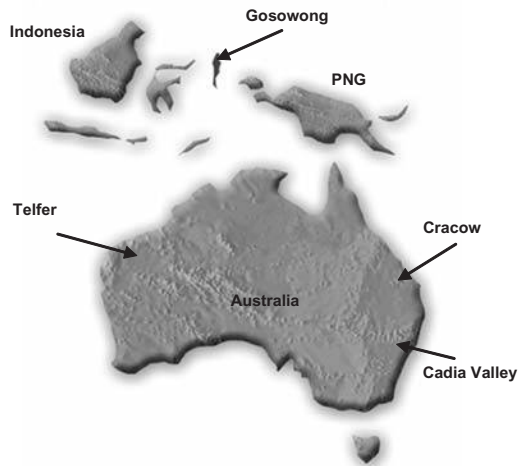


Figure 1. Newcrest Mining Ltd operations in Australia and Indonesia.

Future stages will concentrate on shotcrete support performance evaluation and shotcrete support design guidelines developed on an engineering basis.

2 PRELIMINARY LABORATORY TRIAL

The preliminary phase of the investigation was undertaken in the form of two trials that examined

Table 1. Base mix design for shotcrete used in laboratory.

Ingredient	Quantity (kg/m ³)
Coarse sand	500
Fine sand	455
7 mm Basalt	600
GP cement	380
Fly ash	90
Pozzolith 300 Ri (LRWR)	1200 mL/m ³

the properties of shotcrete during mixing. Both trials were restricted to examining a FRS mix that was *cast* rather than sprayed. The first trial examined the effects of agitation on the effectiveness of a hydration stabiliser (hereafter referred to by its trade name ‘Delvocrete’ from BASF) in a nominal 40 MPa FRS mix (Table 1). The first trial also examined the rate of entrained air loss when an Air Entraining Agent (Microair 940 from BASF, hereafter referred to as ‘AEA’) was added to a shotcrete mix and agitated slowly. All the laboratory trials described in this paper involved use of the same shotcrete mix, listed in Table 1. All laboratory and in-mine trials used the admixtures described above.

The second trial in the preliminary phase of the investigation examined the use of Delvocrete and AEA combined in the same mix. Early-age strength determination was undertaken using a soil penetrometer, needle penetrometer, and beam-end tester according to the methods described by Bernard & Geltinger (2007). The specimens comprised cylinders, cores, panels, and ASTM C116 beams that were cast using the shotcrete mixes produced and examined in the trials.

2.1 Production and curing of specimens

The laboratory investigation proceeded by adding admixtures to a common shotcrete mix to examine their effect on wet characteristics and early-age strength. The two trials each involved delivery of a load of concrete in a large agitator that had been mixed for 20 minutes following addition of 5 kg/m³ of Barchip Shogun fibres. Production of experimental ‘sub-loads’ involved successive discharge of batches of concrete from the large agitator into a mini-agitator resting on a weigh bridge. Using the measured density of the wet concrete, the mass of required concrete was weighed out to within 10 kg. The mini-agitator was then mixed for 10–12 minutes before slump and air content tests began. This method of batching resulted in better than 1% reproduction of concrete volumes, and better than 1% dosage rate of fibre and admixture.

Table 2. Mix details for preliminary trial.

Trial	Sub-load	Admixtures	Agitation
1	1	Delvocrete 3 L/m ³	Slow
	2	Delvocrete 3 L/m ³	No agitation
	3	AEA 300 mL/m ³ , then extra 300 mL/m ³ at 2 hrs	Slow
2	1	No admixtures	Slow
	2	Delvocrete 1.5 L/m ³	Slow

2.1.1 Trial 1 separate dosing of Delvocrete & AEA

The first trial focused on the effects of Delvocrete when dosed into concrete that either stood at rest or was continuously agitated at slow speed. The original 3.0 m³ load was discharged in two separate 1.0 m³ parts and 3 L/m³ Delvocrete was added to each (Table 2). Both mini-agitators were mixed at speed for 10 minutes, and then one continued to agitate continuously at slow speed while the other was stopped from turning completely. The remaining 1.0 m³ of concrete in the original agitator did not have any Delvocrete added but instead was dosed with 300 mL/m³ of AEA and agitated rapidly for 10 minutes and then continuously at slow speed. This was done to check retention of air content with agitation. Cylinders were cast at 1 hour intervals as agitation proceeded.

Air content and slump were measured at regular intervals for all three mixes, and cylinders were taken at regular intervals for the mixes with Delvocrete added. When the concrete began to stiffen and had lost most of the slump, panels and ASTM C116 beams were cast for use in estimation of early-age strength development.

2.1.2 Trial 2 combined dosing of Delvocrete & AEA

For the second trial Barchip Shogun fibres were dosed at 5 kg/m³ and the entire original load of concrete mixed for 10 minutes before discharge into separate mini-agitators was repeated to produce two sub-loads of 1.0 m³. The first of the sub-loads had no admixtures added, and this was slowly agitated for the duration of the trial (Table 2). The second sub-load of concrete had 1.5 L/m³ of Delvocrete and then 600 mL/m³ of AEA added. This was rapidly mixed for 10 minutes, and then slowly agitated for the remainder of the trial.

2.2 Results

2.2.1 Trial 1 separate dosing of Delvocrete & AEA

The results for the first trial indicated that there was only a small difference between the agitated

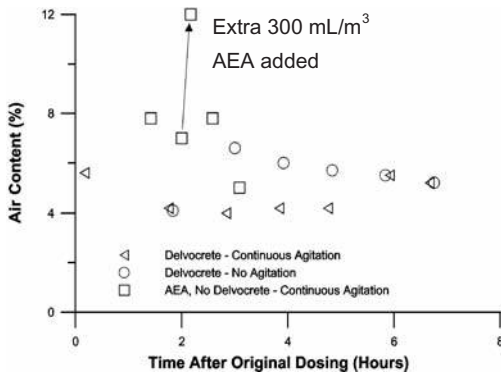


Figure 2. Air content as a function of mixing time after dispensing of concrete in Trial 1.

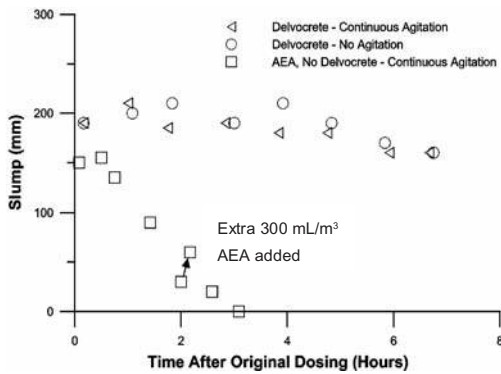


Figure 3. Slump as a function of mixing time after dispensing of concrete in Trial 1.

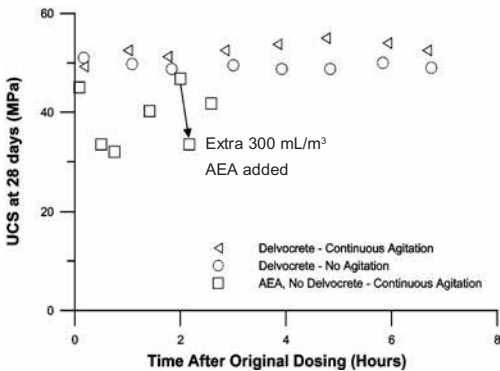


Figure 4. UCS at 28 days as a function of mixing time after dispensing of concrete in Trial 1.

and non-agitated FRS dosed with Delvocrete (shown in Figures 2 and 3). The agitated mix lost slump and gained early-age strength a little faster than the non-agitated load, but the initial difference in air content appeared to be an aberration.

The concrete remaining in the original truck contained AEA but no Delvocrete. This load lost slump and air content quickly, and gained strength at a rate consistent with normal non-accelerated concrete. The AEA caused an initial spike in air content that fell rapidly as agitation continued. Note that this mix originally had 300 mL/m³ AEA added, but after two hours of mixing had a further 300 mL/m³ added to get the total air content up to a desirable level. It should be noted that Delvocrete was added 20 minutes after mixing began and not with the mixing water during batching as per normal practice.

Based on the data generated in this trial, the Delvocrete dosage rate of 3 L/m³ was judged excessive from an experimental point of view (the trial took too long), so future work was planned with a dosage rate of 1.5 L/m³. The present trial nevertheless demonstrated that continuous agitation had only a minor effect on the evolution of properties for FRS compared to concrete stored in a stationary agitator.

The Uniaxial Compressive Strength (UCS) at 28 days appeared to be largely independent of the mixing time after dosing of admixtures (Figure 4) but was strongly affected by the air content of the cast concrete. This conforms to our conventional understanding of concrete technology which maintains that entrained air substantially reduces the strength of concrete. Continued mixing of the concrete in the absence of Delvocrete drove the air out of the mix and hence the UCS rose and fell as air content changed with successive dosing and agitation of the concrete. Slow agitation of the concrete containing Delvocrete appeared to increase the UCS slightly, at least for the first 4 hours after dosing of admixtures (Figure 4).

2.2.2 Trial 2 combined dosing of Delvocrete & AEA

The second trial examined the effect of combined use of Delvocrete and AEA on slump loss and early-age strength gain in cast FRS. When 1.5 L/m³ Delvocrete and 600 mL/m³ of AEA were combined, the air content initially increased to about 16% and then fell and stabilized at around 10–11% for a sustained period (Figure 5). The air content for the last cylinders made was 9%. This behaviour is ideal for the purpose of maintaining a workable slump and using the air as a slump-killer during spraying. In the mix with no admixtures the slump fell rapidly and the mix was unusable after 1½ hours (Figure 5).

When both Delvocrete and AEA were used the slump fell at a moderate rate with mixing and this appeared to be due to steady air loss. The early-age strength was not assessed until 24 hours after mixing began because setting was so slow and

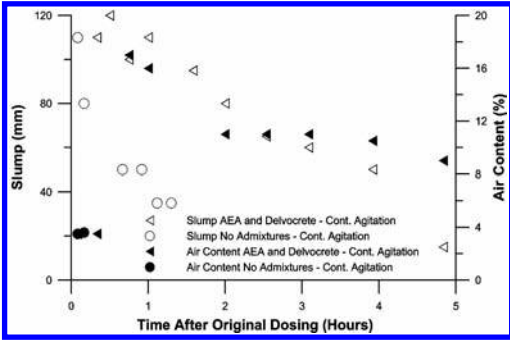


Figure 5. Slump as a function of mixing time after dispensing of concrete in Trial 2.

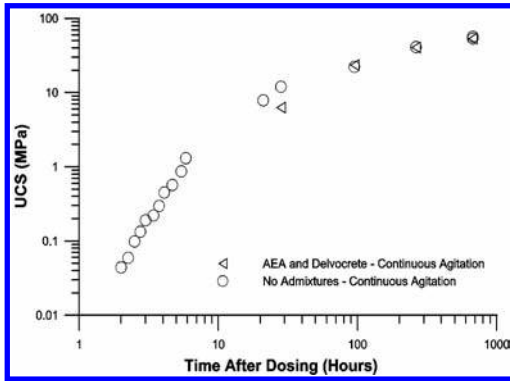


Figure 6. Compressive strength as a function of mixing time after dispensing of concrete in Trial 2.

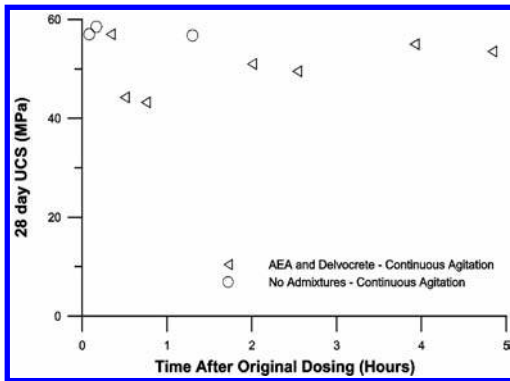


Figure 7. 28 day UCS as a function of mixing time after dispensing of concrete in Trial 2.

occurred over night (Figure 6). By four days age the stabilized mix had caught up in strength development to the mix containing no admixtures. The 28 day UCS broadly reflected the air content of the concrete as mixing proceeded (Figure 7).

The results of this second trial indicated that prolonged mixing slowly drove out the entrained air from the AEA/Delvocrete mix when only 1.5 L/m³ of Delvocrete was used. It appears that the reduction in dosage of Delvocrete between the first and second trials increased the rate at which slump was lost. This appeared to be related to air loss but it remains to be confirmed whether this was the only cause. The data are consistent with previous observations that entrained air will diminish the strength of concrete and that loss of entrained air, either by mixing or spraying, will raise the strength of the concrete when hardened.

2.3 Conclusions

Two important conclusions can be drawn from this first pair of trials concerning the use of admixtures in shotcrete. It is clear that agitating or not agitating a mix with Delvocrete makes no significant difference to slump or early age UCS, nor does it make much difference with respect to air content when 3 L/m³ of Delvocrete is used in the absence of an AEA. Secondly, if an AEA is used without concurrent use of Delvocrete then agitation will drive the entrained air out quickly and result in rapid slump loss. If an AEA is used together with sufficient Delvocrete then air and slump retention are excellent even with continued agitation. This is likely to result in satisfactory spraying properties over a 3–4 hour period after batching.

3 FIRST SPRAYING TRIAL

Following the encouraging results in the preliminary trials, a second series of trials was developed to investigate the influence of two parameters on the performance of shotcrete *as sprayed*. These parameters were: the influence of Delvocrete and AEA on slump and setting characteristics, and the influence of synthetic micro-fibres on early-age behaviour.

Use of AEAs has commonly been claimed to induce segregation in a mix. For this reason, it was intended that part of the FRS used in each of the spraying trials be dosed with micro-synthetic fibres as a means of preventing segregation as well as improving the cohesion of the in-place shotcrete.

3.1 Production and curing of specimens

The trial involved delivery of two loads of shotcrete (see Table 1 for mix design) in a large agitator that had been mixed for 20 minutes following addition of 5 kg/m³ of Barchip Shogun fibres and a base level of admixture (Table 3). Production of the experimental sub-loads involved successive

Table 3. Summary of wet properties and 28 day performance for all sprayed sub-loads (bwc = by weight of cement).

Mix	Details	Slump (mm)	Air (%)	Wet density (kg/m ³)	28d UCS (MPa)		Density (kg/m ³)		Build-up (mm)
					Cylinders	Cores	Cylinders	Cores	
Truck 1	2 L Glenium, 1.5 L Delvocrete	160	9.0	2115	47.3		2150		
Sub-load	0.25 L/m ³ ex. Glenium	190	11.0	2050	39.3		2046		
1	Sprayed out, no SA 160	180	8.0	2168	45.3		2150		20
	SA 160 10.6% bwc								200
	1.5 kg/m ³ microfibre	75	6.0	2197	52.5	57.0	2240	2247	250
Sub-load	0.3 L/m ³ AEA	170	13.5	2011	31.5		2000		
2	Sprayed out, no SA 160	95	6.0	2204	52.8		2210		100
	SA 160 7.6% bwc								220
	1.5 kg/m ³ microfibre	20	8.2	2123	51.8	52.0	2200	2217	300
Sub-load	No extra admixtures	160	11.0	2072					
3	Sprayed out, no SA 160	80	5.0	2204	54.5				60
	SA 160 7.6% bwc								250
	1.5 kg/m ³ microfibre					57.5	2240	2223	250
Truck 2	No Glenium, 1.5 L Delvocrete	20	6.0	2166	54.5		2250		
Sub-load	1.0 L/m ³ AEA	65	13.0	2020	33.3		2060		
4	Sprayed out, no SA 160	20	5.0	2192	53.3		2270		130
	SA 160 4.5% bwc, 5 L water							170	
	0.5 kg/m ³ microfibre	15	4.0	2215	51.5	55.0	2250	2197	120
Sub-load	2.0 L/m ³ AEA	65	16.2	1953	27.5		1990		
5	Sprayed out, no SA 160	20	5.0	2206	50.0		2230		250
	SA 160 4.5% bwc								250
	1.2 kg/m ³ microfibre	15	7.0	2197	45.3	49.8	2250	2203	250+

discharge of 2.0 m³ batches of concrete from the large agitator into a mini-agitator resting on a weigh bridge as per the earlier trials. The mini-agitator was then mixed for 10–12 minutes before slump and air content tests began. A total of five sub-loads were produced with admixture dosages shown in Table 3.

The first three sub-loads were intended to represent typical combinations of admixture and set accelerator used in mining practice. The fourth and fifth sub-loads were modifications of these based on observations of concrete characteristics seen in the first three. The primary variables were the dosage rates of superplasticiser (Glenium 27 from BASF, hereafter referred to as ‘Glenium’), Microair 940 AEA, and synthetic micro-fibres. The set accelerator used for all trials was SA160 from BASF.

Air content tests and slump were measured for both original agitator loads of concrete and all five sub-loads, and cylinders were taken before micro-fibres were added for all the sub-loads. Once micro-fibre had been added and overhead build-up tests completed, panels and ASTM C116 beams were sprayed for use in estimation of early-age strength development. The fourth sub-load specimens were mistakenly sprayed before the microfibre had been added. The use of 2 L/m³ of AEA in the fifth sub-load created a smooth and creamy concrete with very good pumping characteristics but inadequate slump for proper dosing of set accelerator.

3.2 Results

3.2.1 Wet properties

The AEA dosage rate was varied between sub-loads and as a result the air content also differed. Measuring the air content was an important objective during the trials, but unfortunately the meter used for this purpose failed to work at several points in the investigation. Fortunately, the wet density was measured volumetrically as well, and these results provided reliable estimates of air content. Based on air content measurements that were believed to be reliable, the relationship between air content and wet density was used to find that the bulk density of the concrete was about 2300 kg/m³ for an air content of zero, and most of the concrete had a basic air content of 4–5%. This rose when the AEA was added and then fell again after spraying (Table 3). The cylinders produced with the ‘sprayed out’ concrete were made using non-accelerated shotcrete collected after it has been sprayed into a wheelbarrow.

The data in Figure 8 shows that the wet density was closely correlated with the density of the hardened cylinders, but there appeared to be a slight bias toward the hardened density. Additional data from a later trial (Water Compensation Trial, Section 5) is also included in this Figure. The compressive strength appeared to be related to cylinder density (Figure 9) reflecting the fact that the base concrete for all the trials was essentially the same except for changes in air content.

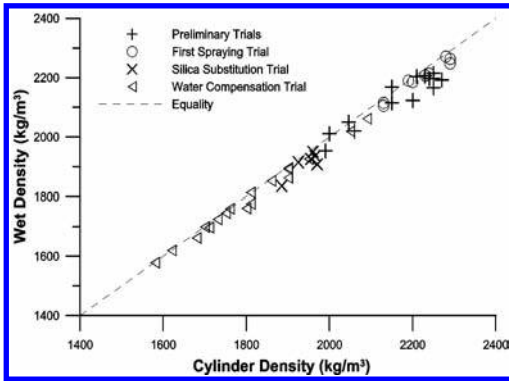


Figure 8. Wet density compared to hardened cylinder density.

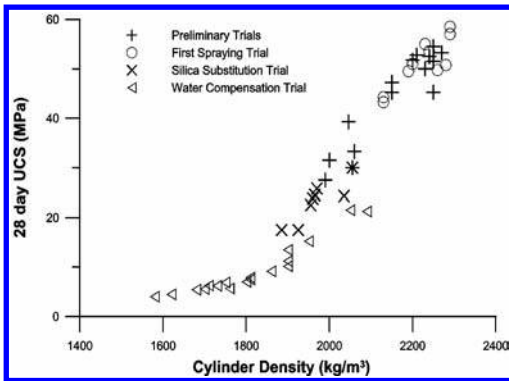


Figure 9. 28 day UCS as a function of cylinder density.

One of the most important issues addressed in this trial was the effect of admixtures on overhead build-up capacity. This was examined using a relative simple technique in which FRS was sprayed upward to produce an inverted cone on an overhead substrate (Figure 10). The height of this cone was measured by driving a steel rule into the concrete near the peak, or by estimation adjacent to the peak.

The build-up capacity was found to vary substantially between sub-loads and with the addition of set accelerator. Micro-fibres also had a measurable effect. The results for all the loads are listed in Table 3. These show that increasing the air content and reducing the dosage of Glenium had the effect of reducing slump immediately after spraying and thereby enhancing build-up capacity even in the absence of the set accelerator. Overhead build-up capacity was observed to improve when AEA was used to replace the Glenium and when higher dosage rates of micro-fibre were added.



Figure 10. Overhead build-up test by spraying.

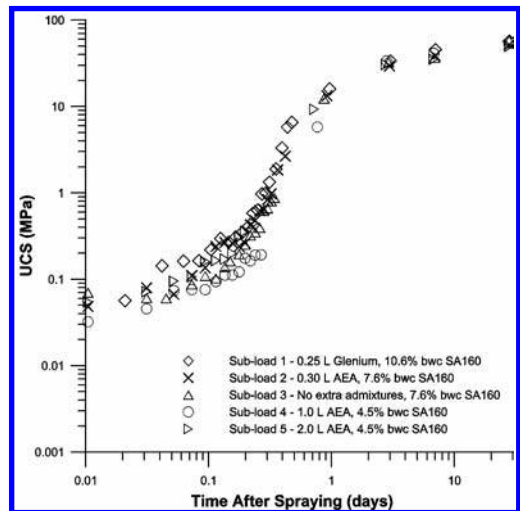


Figure 11. Compressive strength development as a function of time after spraying.

3.2.2 Early strength gain

The rate of early strength gain was an important parameter within this investigation and was therefore examined thoroughly. Using the methods described by Bernard & Geltinger (2007), early-age strength development was measured together with cylinder and core strengths at 3, 7, and 28 days, and are plotted in Figure 11. This shows that while there were substantial differences in the apparent compressive strength at early ages the strength at 28 days was essentially identical for all five sub-loads.

Other influences apparent in Figure 11 included the effect of set accelerator dosage rate on strength gain during the first 4 hours. A dosage rate of SA 160 was at the high end of what is typically used in mining practice (10.6% bwc) and this resulted in a rapid rate of strength increase over the first few hours given that the ambient temperature was only 16°C. When the dosage rate was dropped to 7.6% bwc the rate of early strength gain correspondingly fell, and when it was dropped to 4.5% bwc the rate of strength gain was quite modest. The reason the set accelerator rate was decreased was that the AEA and microfibre appeared sufficient to maintain adequate build-up capacity overhead at this moderate dosage.

Sub-loads 4 and 5 appeared to demonstrate the importance of the micro-fibre to cohesion and build-up capacity more than anything else. Sub-load 4 contained only 0.5 kg/m³ of micro-fibre and achieved an overhead build-up capacity of only 120 mm (Table 3). Increasing the dosage rate to 1.2 kg/m³ in sub-load 5, plus the use of a lower slump concrete, improved the build-up capacity to 250 mm even without set accelerator. Addition of set accelerator improved the stability of overhead build-up to the point where no concrete fell even 20 minutes after spraying.

3.3 Conclusions

Several important conclusions can be drawn from this trial concerning the use of admixtures in shotcrete. The trial established that the Glenium content of FRS can be reduced and replaced by an AEA if used in combination with Delvocrete. The loss of air that occurs when heavily air-entrained shotcrete is sprayed has a slump-killing effect (Jolin & Beaupré, 2000, 2004) that strongly enhances build-up capacity on over-head surfaces. Synthetic micro-fibres provide the further benefit of increased internal cohesion within the concrete leading to superior build-up capacity.

When used in combination with a low dosage rate of set accelerator, a shotcrete dosed with AEA, Delvocrete, and synthetic micro-fibre demonstrated greater than 250 mm of overhead build-up capacity. This was substantially better than was possible using a similar mix dosed with Glenium and Delvocrete alone. A dose rate of 0.5 kg/m³ micro-fibre did not appear sufficient to induce good build-up capacity. Similarly, it is recommended that a higher slump be used in future prior to addition of the AEA.

4 SILICA SUBSTITUTION TRIAL

The third laboratory trial was intended to examine whether alternatives to silica fume could be

found that resulted in similar build-up capacity and early-age strength gain characteristics but at a lower price. These alternatives were: Betocarb 1 micro-ground calcium carbonate, Hi-pozz engineered fly-ash, Microsilica 600 volcanic-sourced silica, Zeolite powder, and Ground Granulated Blast Furnace Slag (GGBFS).

Based on results obtained earlier, AEA was used to replace the Glenium in all mixes produced during the trial and micro-fibres were added at the end of each sub-load to examine their influence on overhead build-up capacity. An alkali-free set accelerator (SA 160 from BASF) was used to accelerate hydration, and build-up capacity was examined both with and without SA160 set accelerator. The slump was adjusted to a target of 110 mm prior to spraying of each sub-load of concrete.

4.1 Production and curing of specimens

The silica substitution trial included the production of panels and other specimens using the same nominal 40 MPa shotcrete mix presented in Table 1. The trial involved delivery of two loads of concrete in a large agitator that had been mixed for 20 minutes following addition of 5 kg/m³ of Barchip Macro fibres and a base level of admixture comprising Delvocrete (1.5 L/m³) and Microair 940 AEA (500 mL/m³). The slump of the concrete in these trucks was adjusted to 120 mm using water. Production of the experimental 'sub-loads' involved successive discharge of 1.5 m³ batches of concrete from the large agitator into a mini-agitator resting on a weigh bridge. A total of 30 kg/m³ of either silica fume or one of the silica substitutes was added to each sub-load during dispensing (Table 4). The mini-agitator was then mixed for 5 minutes before slump and density tests began. Most of these substitutes exhibited some degree of pozzolanic activity; only the Betocarb CaCO₃ lacked pozzolanic properties. The effective binder content of each mix therefore varied slightly.

The variable altered between each sub-load was the silica component or substitute in accordance with Table 4. The slump fell as the dry powder was added, so an additional 500 mL/m³ of AEA was added to each mini-agitator. Some additional water was also required for most of the loads because of the high water demand of the additives, but the amount varied.

Density and slump were measured for both original agitator loads of concrete and all six sub-loads, and cylinders were taken before micro-fibres were added for all the mixes. ASTM C116 beams were sprayed for use in estimation of early-age strength development before micro-fibres were added to the concrete. After micro-fibre had been added to each

Table 4. Summary of mix details for all loads and sub-loads investigated in silica substitution trial.

Mix	Details
Truck 1	1.5 L/m ³ Delvocrete and 500 mL/m ³ AEA
Sub-load 1	500 mL/m ³ extra AEA, 30 kg/m ³ CSF, 1.3 L/m ³ water, then 1.5 kg/m ³ microfibre
Sub-load 2	500 mL/m ³ extra AEA, 30 kg/m ³ Betocarb, 2.7 L/m ³ water, then 1.5 kg/m ³ microfibre
Sub-load 3	500 mL/m ³ extra AEA, 30 kg/m ³ Hi-Pozz, 3.3 L/m ³ water, then 1.5 kg/m ³ microfibre
Truck 2	1.5 L/m ³ Delvocrete and 500 mL/m ³ AEA
Sub-load 4	500 mL/m ³ AEA, 30 kg/m ³ Microsilica 600, 0.7 L/m ³ water, & 1.5 kg/m ³ microfibre
Sub-load 5	500 mL/m ³ AEA, 30 kg/m ³ Zeolite, 3.3 L/m ³ water, & 1.5 kg/m ³ microfibre + 10 L/m ³
Sub-load 6	500 mL/m ³ extra AEA, 30 kg/m ³ GGBFS, 3.3 L/m ³ water, then 1.5 kg/m ³ microfibre

sub-load an additional overhead build-up test was conducted.

4.2 Results

4.2.1 Wet properties

The AEA dosage rate and Delvocrete content were identical for all the loads and sub-loads, but there were slight differences in water content due to the varying water demand of the added silica or substitute. The target slump was 110 mm, but some difficulty was encountered in achieving this in the latter part of the day due to the high ambient temperature and water demand, particularly for the Zeolite. A decision was made to keep the water/binder ratio relatively constant for all sub-loads, hence the slump varied.

The air temperature increased from about 24°C during the first trial to about 32°C for the last one. It was observed that the average density of the concrete fell during the day, possibly because the mini-agitator was agitated for a longer period of time while preparations were made for the next session.

Most of the sub-loads had similar wet properties, but the Betocarb was significantly better in the wet state than the rest due to the creamy texture and excellent finishing characteristics. Pumping also appeared to be easier with this additive included. The overhead build-up capacity varied significantly between sub-loads, as shown in Table 5. For the Condensed Silica Fume (CSF), the build-up capacity was moderate prior to addition of micro-fibres. However, with micro-fibres the build-up was outstanding and the overhead

Table 5. Summary of wet properties and overhead build-up capacity for all mixes.

Mix	Slump (mm)	Air (%)	Wet Den. (kg/m ³)	Build-up (mm)			
				No Microf.		Microfibre	
				No Acc	Acc	No Acc	Acc
Truck 1	120	2*	2198				
CSF	110	13.4	1950	100	150	260	300
Betocarb	100	14.9	1915	120	270	270	290
Hi-pozz	110	18.6	1835	150	240	120	150
Truck 2	120	2*	2203				
M600	90	15.3	1908	230	250	270	180
Zeolite	60	14.0	1936	230	150	180	200
GGBFS	90	14.5	1926	220	150	250	200

* Estimated for original concrete as delivered, zero air density taken to be 2245 kg/m³.

mounds stayed in place well after spraying stopped (Figure 10). For the Betocarb, the build-up capacity was excellent but adhesion to the surface was problematic, probably due to surface dryness. The mounds fell off after 5 minutes through adhesive failure at the boundary with the culvert invert. For the Hi-pozz, the build-up capacity with no micro-fibre was similar to that achieved with the Betocarb, but when micro-fibres were added there appeared to be an excessive amount of layering of accelerator during spraying and the mound fell off due to cohesive failure, possibly at one of the accelerator layers. This pattern was repeated for several of the following sub-loads as the slump steadily fell due to high temperatures and increased water demand. This indicated one of the problems encountered when spraying low slump shotcrete together with set accelerator. The best build-up capacity was achieved without set accelerator for sub-loads 4 to 6, largely due to the adverse effects associated with layering of set accelerator in the other sub-loads.

From the results of this trial, and earlier experience with trials conducted for other companies, it appears that cohesion falls as the slump of the concrete increases, and that improved cohesion can more readily be achieved with micro-fibres rather than mineral additives. Cohesion appears to be indirectly affected by water demand so materials with a high water demand degrade performance with respect to build-up capacity. However, adhesion to the substrate is not affected by the presence of micro-fibres. Adhesion appears to be primarily affected by the slump of the concrete, dryness of the surface, and the mineral composition of the paste. The greatest overhead build-up possible appears to be about 300 mm (equivalent to 6–7 kPa

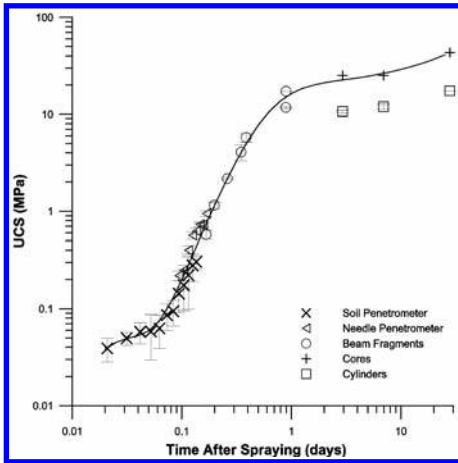


Figure 12. Compressive strength development as a function of time after spraying for sub-load 3 with Hi-pozz.

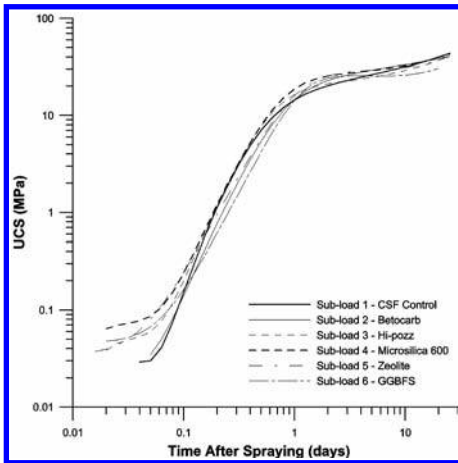


Figure 13. Compressive strength development as a function of time after spraying for all sub-loads.

tensile stress) as this level was never exceeded in all the trials conducted.

4.2.2 Early strength gain

The rate of early-age strength gain is an important issue due to its influence on rate of excavation during heading development. Using the methods described by Bernard & Geltinger (2007), early-age strength development was measured together with cylinder and core strengths at 3, 7, and 28 days. Results are plotted for the Hi-pozz additive in Figure 12 and all the trends have been combined in Figure 13. This shows that the general pattern of strength development was very similar for all six sub-loads; only the magnitudes of the indirect UCS over the first 2 hours differed significantly.

The cause of this early-age difference in strength was probably the ambient temperature as the pattern of strength increase broadly matched the sequence of spraying (increasing from sub-load 1 through to 6). The slump was also higher for the earlier mixes. The cast cylinder strengths were much lower than that of the cores because the air contents of the former were much greater (Tables 5 & 6). Taking into account the temperature, the best performing sub-loads over the first 12 hours appeared to be the sub-loads containing Microsilica 600 and Hi-pozz.

4.3 Conclusions

This investigation demonstrated that all of the mineral additives selected could be used as satisfactory substitutes for silica fume with only a minor change in early-age strength gain. The GGBFS was the only silica substitute that reduced the 28 day UCS by a significant amount. Build-up capacity appeared to be more strongly affected by the additives used, with the Microsilica 600 producing the best build-up capacity before micro-fibre was added. However, micro-fibres increased cohesion

Table 6. Summary of hardened properties for all mixes.

Mix	Additive	Wet concrete		Cylinders		Cores		28 day UCS	
		Density (kg/m ³)	Air (%)	Density (kg/m ³)	Air (%)	Density (kg/m ³)	Air (%)	Cylinder (MPa)	Cores (MPa)
Truck 1		2198*	2*	2035	9.4§			24.3	
Sub-load 1	CSF	1950	13.4	1960	12.7	2200	2.0	23.8	46.0
Sub-load 2	Betocarb	1915	14.9	1925	14.3	2210	1.6	17.5	41.0
Sub-load 3	Hi-pozz	1835	18.6	1885	16.1	2210	1.6	17.5	43.5
Truck 2		2203	2*	2055	8.5§			30.0	
Sub-load 4	Microsilica 600	1908	15.3	1970	12.2	2193	2.4	25.8	45.0
Sub-load 5	Zeolite	1936	14.0	1965	12.5	2190	2.4	24.5	45.3
Sub-load 6	GGBFS	1926	14.5	1955	13.0	2140	4.7	22.5	36.5

* Estimated for original concrete as delivered, § Based on wet density of 2245 kg/m³.

and build-up capacity more effectively than any of the mineral additives. If micro-fibres are used in a mix, then the choice of silica additive or substitute is largely inconsequential as far as build-up capacity is concerned. This trial therefore established that shotcrete users have a wide range of effective options to choose from when it comes to silica additives and should not assume that Condensed Silica Fume is the only acceptable silica additive necessary for satisfactory spraying and early-age strength development characteristics.

5 WATER COMPENSATION TRIAL

In the trials conducted above it was noted that the addition of synthetic micro-fibres reduced the slump of the concrete as delivered and caused problems with pumping and dosing of set accelerator. It was therefore necessary to investigate how much water is required to compensate for the slump loss associated with use of synthetic micro-fibres. A simple mixing and casting trial involving the addition of varying doses of synthetic micro-fibre to three agitator-loads of shotcrete was therefore undertaken.

The investigation involved the execution of slump tests and production of cast cylinders using the shotcrete mix listed in Table 1. Three separate loads of concrete were delivered in mini-agitators that had been mixed for 10 minutes following addition of 5 kg/m³ of Barchip Shogun fibres and a base level of admixture comprising Delvocrete (1.0 L/m³) and Microair 940 AEA (1.0 L/m³). The slump of the concrete in these trucks was adjusted to a target of 170 mm using water. The wet density was measured prior to addition of the admixtures to establish the base density of the concrete.

The dosage rate of synthetic micro-fibre added to each truck varied from 0.67 to 1.5 kg/m³. The micro-fibre used was a 12 mm long ~32 micrometre diameter polypropylene fibre. The first load of concrete and set of cylinders (Set 1, Table 7) was dosed with 0.67 kg/m³ of micro-fibre, the second with 1.1 kg/m³ micro-fibre, and the third with 1.5 kg/m³ micro-fibre. A slump test and a pair of cylinders were produced after the initial mixing with admixtures was completed. This established the benchmark slump that was required to be restored after micro-fibre had been added and these specimens were labelled 'A' for later reference (Table 7). The micro-fibre was then added and the load mixed for 5 minutes, after which a slump test and pair of cylinders labelled 'B' were cast. Additional successive amounts of water (3 or 5 L at a time) were then added to each load and slump tests were conducted and cylinders made. These became specimens 'C' through to 'F'. Note that approxi-

Table 7. Summary of wet and hardened properties for all mixes, 0.5 m³ delivered for each load.

Set	Addition	Slump (mm)	Wet D. (kg/m ³)	Air (%)	UCS (MPa)	Dry D. (kg/m ³)
1A	Admix.	165	2019	9.6	21.5	2050
1B	Microfibres	160	1926	13.8	15.3	1950
1C	3 L water	165	1891	15.3	11.3	1900
1D	5 L water	180	1813	18.8	7.5	1810
1E	5 L water	210	1760	21.2	5.7	1760
1F	5 L water	215	1699	23.9	5.6	1700
2A	Admix.	170	2061	7.7	21.3	2090
2B	Microfibres	155	1896	15.1	13.5	1900
2C	5 L water	160	1862	16.6	10.2	1900
2D	5 L water	175	1776	20.5	7.9	1810
2E	5 L water	210	1761	21.1	7.1	1800
2F	5 L water	220	1724	22.8	6.2	1730
2A	Admix.	180	1852	17.1	9.2	1860
2B	Microfibres	145	1745	21.9	6.9	1750
2C	5 L water	155	1696	24.1	6.3	1710
2D	5 L water	165	1662	25.6	5.5	1680
2E	5 L water	170	1620	27.5	4.5	1620
2F	5 L water	190	1578	29.4	4.0	1580

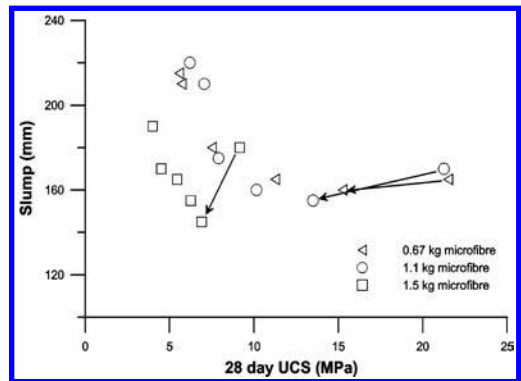


Figure 14. Relation between slump and 28 day UCS of concrete. Arrows indicate the change that occurred when synthetic micro-fibres were added to each mix (probably due to extra air entrainment).

mately 30–40 kg of concrete were discharged from each truck for each set of tests and cylinders, thus the original 0.5 m³ of concrete was progressively diminished in volume. It is estimated that by the last tests only 0.35–0.4 m³ remained.

The change in slump and UCS as water was progressively added to each mix is shown in Figure 14. The arrows indicate the change in properties that occurred after addition of the micro-fibres. Slump fell with addition of the micro-fibres, particularly when the 1.5 kg/m³ dosage rate was employed. However, it was apparent that air content also increased as the micro-fibres were added (Table 7). Micro-fibres are not known for their air-entraining

properties so this suggests that prolonged mixing increased the air content. Noting that air content continued to increase for specimens B through to F even though no additional AEA was added indicates that the amount of air entrained in a mix appeared to be controlled either by the duration of mixing or the amount of water available in the paste.

Whichever was the cause of the increased air content, the slump steadily recovered as more water was added. Based on the data in Table 7, and compensating for the progressive loss of concrete from each bowl during the trial, an addition of approximately 6 L/m³ water was all that was required to compensate for 0.67 kg/m³ of micro-fibre addition. The amount of water required to compensate for 1.1 kg/m³ of micro-fibre was about 15 L/m³, and about 35 L/m³ was required for 1.5 kg/m³ of micro-fibre. However, these estimates will have been corrupted by the parallel influence of increased air content and will require checking in the field. It is possible that the more limited period of mixing typical of production concrete will necessitate more water (or AEA/ superplasticiser) to achieve the required slump when micro-fibre is included.

6 IN-MINE SPRAYING TRIAL

To complement the laboratory shotcrete trials described above, and implement the knowledge gained through these trials in a production environment, a series of shotcrete trials were undertaken within the Ridgeway Deeps mine (for Newcrest Mining Ltd).

The Ridgeway Deeps mine (RWD) is located in the Cadia Valley approximately 30 km south of Orange in NSW, Australia (Figure 1). Ridgeway Deeps is a new underground block cave mine and is currently being constructed at a depth of 1,100 m below surface. It is situated below the existing Ridgeway gold mine which uses the sub-level caving method.

The purpose of the mine trials was to examine whether cost benefits identified through mix design changes developed in the laboratory-based shotcrete trials could be translated into similar benefits within Newcrest operations. These cost benefits were based on the replacement of superplasticiser by an AEA to achieve suitable workability, and the elimination of silica fume from the mix. The use of synthetic micro-fibres to increase cohesion was also examined. The mix used in the laboratory-based trials differed in several ways from the shotcrete used at RWD, hence it was necessary to undertake on-site trials to confirm that the favourable results obtained in the laboratory could be reproduced in the field.



Figure 15. Spraying test site at ridgeway deeps.

The laboratory trials had established that the currently used superplasticiser, Glenium 27, could be replaced by an Air Entraining Agent, Micro-air 940, to simultaneously achieve both improved pumping and a substantial reduction in admixture costs. The introduction of about 10–14% entrained air by volume created a workable mix by effectively increasing the paste fraction within the shotcrete. This fluidizing fraction was then displaced during the spraying process so that the slump fell back to the level evident before the AEA was introduced. The result was a stiffening action that complemented the stiffening associated with calcium sulfo-aluminate hydrate growth caused by the introduction of set accelerator at the nozzle. In theory, the dosage rate of set accelerator can therefore be reduced to achieve a further saving in overall costs.

6.1 Production and spraying of shotcrete

The in-mine trials involved a sub-set of investigations based on the most favourable results achieved in the laboratory. The November 2008 trials were based on modifications to the existing FRS mix design used at RWD. The first trial involved elimination of Glenium 27 superplasticiser and its replacement with Microair 940 AEA; the second trial involved the additional elimination of one half of the MS695 liquid micro-silica (with no compensation in cement content); and the third involved the complete elimination of MS695 from the mix.

In each of the trials a load of 5 m³ was batched according to the ingredients in Table 8 at the Boral pre-mix plant on the surface with 7 kg/m³ of Bar-chip Shogun fibres and no initial AEA. The slump was deliberately kept low so that workability did

Table 8. Mix designs used for Nov 2008 in-mine spraying trials.

Ingredient/Parameter	Quantity (kg/m ³)		
	Trial 1	Trial 2	Trial 3
Coarse aggregate (10 mm)	566	566	567
Washed sand	1106	1108	1110
Cement (Type GP)	392	391	391
Fly ash	55	55	55
MS 695 silica fume (Litres)	7	3.5	0
Microair 940 AEA	1000 mL	1000 mL	1000 mL
Delvocrete Hydration Stab.	2000 mL	2000 mL	2000 mL
Slump at batch plant (mm)	55	75	70
Slump underground (mm)	–	100	35
Slump with AEA (mm)	110	160	110
Slump, 1.5 kg/m ³ microfibres	80	–	–
Density without AEA (kg/m ³)	2309	2310	2310
Density with AEA (kg/m ³)	2058	1963	–
Air content with AEA (%)	13.5*	17.5*	–

* Calculated zero-air shotcrete density 2380 kg/m³.

not become excessive when the AEA was added later. The slump and wet density were measured before the truck drove down to the 4804 Undercut level where slump was again measured upon arrival. AEA was then added and the concrete agitated vigorously for 5 minutes. An initial spray onto the backs was performed, and then ASTM C1550 toughness panels, cores, and early-age test specimens were sprayed. Synthetic micro-fibres at a dosage rate of 1.5 kg/m³ were then added, the slump was again tested, and the rest of the concrete was sprayed onto the backs to examine stickiness.

6.2 Shotcrete performance

The first underground mix sprayed satisfactorily without any Glenium being present in the mix. The slump of the concrete going into the pump was satisfactory at 110 mm, but much of this was lost during spraying which was beneficial to the cohesion of the applied shotcrete. The concrete displayed negligible fall-outs soon after spraying. When the micro-fibres were added, the slump fell to a level that was less than optimal, but spraying was still satisfactory and no fall-outs of fresh concrete were seen on the backs. Note that the early-age strength data shown in Figure 16 for this mix was for shotcrete sprayed prior to the micro-fibres being

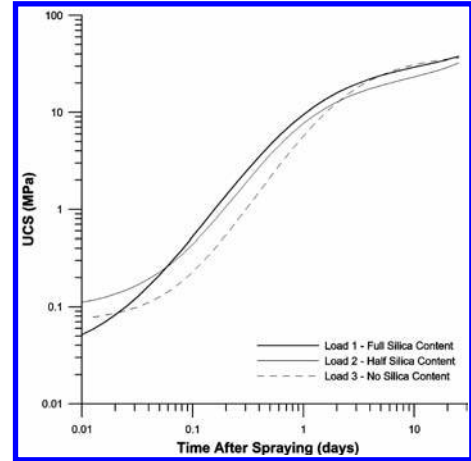


Figure 16. Early-age strength for all three mixes sprayed in the underground trials.



Figure 17. ASTM C-1550 toughness panels being sprayed at underground test site.

added. The same is true for the second and third trials. This trial confirmed that Glenium 27 could be replaced by Microair 940 with no deterioration in workability, pumpability, or spraying characteristics. Early age strength was determined using a soil penetrometer, needle penetrometer, and beam-end tester (Bernard & Geltinger, 2007), and later age strengths were determined using cores. The rate of strength gain was quite satisfactory, and late age strength was within a satisfactory range (Figure 16). The ambient temperature during the trial was 34°C dry bulb and 31°C wet bulb.

The second load of concrete was very similar to the first in containing no Glenium, but the dosage rate of MS695 liquid silica was reduced to only 3.5 L/m³. Unfortunately, the truck driver mistakenly added water to the mix (20 L) upon arrival at

Table 9. Hardened properties for the three RWD trials conducted November 2008.

Parameter	Performance		
	Trial 1	Trial 2	Trial 3
Core UCS 7 days (MPa)	26.7	21.0	26.3
Core UCS 28 days (MPa)	39.3	34.2	36.7
Cylinder UCS 7 days (MPa)	21.0	13.5	22.0
Cylinder UCS 28 days (MPa)	27.0	18.0	26.3
Energy Absorption, 28 days, ASTM C1550 40 mm (J)	441	389	457
Pumping characteristics	Good	Good	Good
Spraying characteristics	Good	Good	Good
Fall-out incidence	Okay	Inferior	Okay

Note: The toughness specified by the mine for the Nov 2008 trials was 360 Joules.

the Undercut level and the slump was excessive. The strength and toughness of the second set were therefore poorer than that of the other sets. The spraying characteristics of this mix were similar to that of the first, but fall-outs were more frequent. Addition of micro-fibres reduced fall-outs, but the mix was essentially too wet to make a direct comparison to the first mix. The initial rate of strength gain was similar to that of the other two mixes but the final strength was substantially lower for both the cores and cylinders (Table 9).

The third load contained no Glenium and no MS695 micro-silica. The slump was correctly maintained at about 70 mm prior to driving down to the Undercut level, and upon arrival this had fallen to about 35 mm. The AEA and 25 L of water were added to compensate for the low slump, and this brought the slump up to 110 mm which was considered ideal. The spraying characteristics of this mix were quite satisfactory, indicating that the MS695 appeared superfluous to requirements. Fall-outs were minimal even before the micro-fibres were added. Comparing the performance of this mix to the second suggested that slump had a more significant influence on fall-outs than silica content. In the presence of an AEA, the silica content appeared to have little influence on stickiness and resistance to fall-out.

The early-age performance data presented in Figure 16 indicates that the rate of strength development fell as the silica content was progressively reduced. This is consistent with a steady fall in total cementitious content and relative increase in water/binder ratio and may possibly have to be compensated for through an increase in cement content.

6.3 Ongoing mine trials

Further trials were undertaken under operational conditions at Ridgeway Deeps mine in May, June &

Table 10. Mix design used for May-July 2009 mine production spraying trials.

Ingredient/Parameter	Type B mix
Coarse aggregate (10 mm)	570
Washed sand	1095
Cement (Type GP)	400
Fly ash	55
Microair 940 AEA	1000 mL
Delvocrete Hydration Stab.	2000 mL
Slump at batch plant (mm)	55
Slump with AEA (mm)	100–120

Table 11. Hardened properties for the RWD production trials conducted May-July 2009.

Parameter	Performance		
	May	Jun	Jul
Core UCS 7 days (MPa)	20.0	29.0	31.0
Core UCS 28 days (MPa)	34.5	38.3	38.3
Cylinder UCS 7 days (MPa)	25.0	25.0	23.0
Cylinder UCS 28 days (MPa)	32.3	32.0	35.5
Energy Absorption, 7 days, ASTM C1550 40 mm (J)	348	520	513
Energy Absorption, 28 days, ASTM C1550 40 mm (J)	411	577	565

July 2009. The 4804 Undercut Level, situated over 1,000 m below surface, was again used for the trials. One concrete agitator truck was fitted with an on-board dosing pump to allow the truck operator to add the AEA remotely from ground level. The AEA was added into the mix once the truck had reached the spraying site. The travel time for a concrete truck from the batch plant to the underground spraying site is approximately one hour.

For the particular areas sprayed for these trials, the mix design was as per Table 10, with 9 kg/m³ of Barchip Shogun fibres added to achieve a target toughness of 500 Joules.

The strength and energy absorption results for the three trials are shown in Table 11. Ongoing trials continue at the time of writing. Pumping and spraying characteristics of the mix have remained good with no negative reports regarding fall-out or patchy strength development incidence. The concrete agitator truck operators have also found the on-board dosing pump useful.

7 ECONOMIC ASSESSMENT

The savings in cost per supplied cubic metre of shotcrete for alternative combinations of additives have been calculated against the RWD base case (Table 12).

Table 12. Cost reductions in materials.

Additive combination	Savings*
AEA added, No Glenium, 100% MS 695 No microfibres	2.7%
AEA added, No Glenium, 50% MS 695 No microfibres	11.4%
AEA added, No Glenium, No MS 695 Microfibres added	18.2%
AEA added, No Glenium, No MS 695 No microfibres	20.0%

* Savings compared to base case site mix design (per cubic metre supplied).

In the case of Ridgeway Deeps, the above figures show that significant savings are possible with the use of alternative additives. As each mine site is unique with different raw materials, batch plants, travel times and spraying conditions, the above combinations may not be suitable for all FRS and the savings will be different. For NML, the next stage will include testing some of these combinations at other underground sites. In terms of extra costs associated with switching admixtures, the only one identified to date is the addition of a dosing pump to the concrete agitator truck at a cost of A\$500 to allow addition of AEA at the underground spray site.

8 CONCLUSIONS

Several important conclusions can be made following this investigation. It is clear that numerous admixtures are available on the market and that these help to improve the performance of FRS. However, conventional practice and wisdom within the shotcrete industry does not appear to make the most effective and economical use of available admixtures.

This series of trials has demonstrated that expensive admixtures such as polycarboxylic superplasticisers are often used in place of much cheaper and more suitable admixtures available from the same supplier. Lateral thinking, rational assessment of the costs and benefits of each available admixture, and carefully executed trials can establish that an

Air Entraining Agent combined with a hydration stabilizer is a more attractive solution for FRS workability when used for ground support in mines than superplasticisers. Condensed Silica Fume is also just one of many options available for enhancement of spraying characteristics, but in some circumstances it may be found that excellent spraying can be achieved without the need for any type of silica additive.

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REFERENCES

- American Society for Testing and Materials standard C-1550, "Standard Test Method for Flexural Toughness of Fiber Reinforced Concrete (Using Centrally Loaded Round Panel)", ASTM, West Conshohocken, PA, 2008.
- ASTM C 116 (Reapproved 1980) "Standard Test Method for Compressive Strength of Concrete Using Portions of Beams Broken in Flexure", American Society for Testing and Materials, 1968.
- Australian Standard AS1012 *Testing Concrete*, Standards Australia, Sydney.
- Bernard, E.S., 2008a. "Trial 1—Preliminary Tests", *TSE Report 191*, August.
- Bernard, E.S., 2008b. "First Shotcrete Trial", *TSE Report 192*, August.
- Bernard, E.S., 2008c. "Ridgeway Deeps In-mine Shotcrete Trial", *TSE Report 193*, December.
- Bernard, E.S. & Geltinger, C., 2007. "Early-age determination of compressive strength in shotcrete", *Shotcrete*, Vol. 9, No. 4, pp. 22–27, American Shotcrete Association.
- Jolin, M., & Beaupré, D., 2000. "Temporary high initial air content wet process", *Shotcrete*, Vol. 2, No. 1, pp. 32–34, American Shotcrete Association.
- Jolin, M., & Beaupré, D., 2004. "Understanding wet-mix shotcrete; mix design, specifications and placement", in Potvin, Stacey, & Hadjigeorgiou (Eds.), *Surface Support in Mining*, pp. 263–267, ACG, Perth, Australia.

Advances in shotcrete education and nozzleman certification in America

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ABSTRACT: Since its inception in 1998, the vision of the American Shotcrete Association (ASA) has been to promote the safe and beneficial use of the shotcrete process as a viable option for conventionally placed (formed) concrete, or as stated on their website; “to have the shotcrete process understood and used in every beneficial application”. Through their mission, the ASA has become a clear and consistent voice for the shotcrete industry. During the past decade, ASA has sponsored or overviewed over 400 shotcrete educational sessions. They have spread the word about the shotcrete process through the publication and distribution of Shotcrete Magazine, which today has over 10,000 subscribers in 92 countries around the world. Through these initiatives, ASA has been successful in educating engineers, contractors and government authorities about the benefits of the shotcrete process, and the better these sectors of the construction industry understand shotcrete, the more the shotcrete industry will have the opportunity to grow.

This paper provides an overview of both ASA's shotcrete education programs, including the new shotcrete educational program on underground mining shotcrete, and the ACI certification program, including the new educational documents recently published.

1 INTRODUCTION

The American Shotcrete Association was founded in 1998 to promote shotcrete with the Vision Statement: “to have the shotcrete process understood and used in every beneficial application”, and the Mission Statement: “to encourage and promote the safe and beneficial use of the shotcrete process”.

With this in mind, the discussion over the importance of shotcrete nozzleman certification at the time, and the vehicle by which a certification program could be delivered, was debated by industry members everywhere, and lead to one of the ASA's first industry tasks, the offering of a shotcrete nozzleman certification program. This program, based on the initial work of ACI C660 Committee, was proposed by the ASA with the hope to fast track its acceptance by the American Concrete Institute. The effort was successful as ACI adopted the ACI shotcrete nozzleman certification program as part of its certification activities (Dufour 2008; Isaak 2002).

The volunteer members of the ASA's first Certification Committee worked diligently to offer a Shotcrete Nozzleman Certification program and launched the first shotcrete Training and Certification session in 1999. This session also served to

qualify future Trainers and Examiners and helped define what eventually became two separate entities; the ASA's training program requiring ASA trainers and ACI's Shotcrete Nozzleman Certification program requiring ACI Examiners. As a result, the ASA's certification effort ran until 2001 when the ACI C660 Committee introduced the first ACI Shotcrete Nozzleman Certification program. The ASA certification committee then became the Education Committee (Morgan & Dufour 2008).

Since then, the ASA has recognized the need for a comprehensive education program for nozzlemen who plan on taking the ACI certification examination. Thus, an early priority of the ASA Education Committee was the development of a Shotcrete Nozzleman Education program. This program was based on development of PowerPoint CD Education modules for use in classroom instruction, but also included hands-on training in the field in either the wet or dry-mix shotcrete process, or both. As a result, the ASA now acts as a Local Sponsoring Group to administer the ACI examination. The ASA also provides Shotcrete Education for nozzleman taking the ACI Certification Examination or for those simply interested in learning correct nozzleman techniques. It should be noted that a

500 hours of nozzling experience is required to qualify for shotcrete nozzleman certification.

In 2007, the ASA Underground Committee developed an underground mining shotcrete education module designed to educate those involved in the mining and tunneling industries on the beneficial and safe use of shotcrete as a concrete placement method for ground support and underground construction. The Committee surveyed the need for *underground shotcrete nozzleman certification* and concluded that proper education and training was more of a priority than certification in the underground world. Today, most underground contractors and mining companies have their own personnel trained about the safe and proper placement of shotcrete for underground construction, stabilization and support, using their own shotcrete placement equipment.

2 SHOTCRETE NOZZLEMAN EDUCATION PROGRAMS

The Education and Underground Committees of the American Shotcrete Association have developed a series of Shotcrete Education Modules segmented to best suit the area in which shotcrete education takes place. This ASA Education program consists of concrete/shotcrete related technical modules such as: introduction to shotcrete, history and uses of shotcrete, shotcrete materials and properties, shotcrete mix designs, quality control/quality assurance, shotcrete equipment, preparation for shotcreting, shotcrete nozzling and application techniques, shotcrete finishing and curing procedures, and safety. Modules for shotcrete for swimming pools and spas are also available.

2.1 Shotcrete education

Shotcrete education would not be possible without an adequate number of ASA approved Educators. These Educators, currently ASA members, have been approved by the American Shotcrete Association's Education Task Group. To qualify as an ASA Educator, the ASA has strengthened the requirements and must have:

- Sufficient shotcrete knowledge and work experience and meet a minimum of at least four (4) out of the nine (9) following qualification requirements:
- Minimum of five (5) years experience in the shotcrete construction field as a supplier with service exposure, as an engineer or technician servicing shotcrete equipment.
- Minimum of one (1) full year of experience in shotcrete application as: an instructor, service

technician for shotcrete materials or equipment related to shotcrete.

- Safety training at a minimum of five (5) civil or mining shotcrete projects.
- Minimum of five (5) years experience as a shotcrete nozzleman in dry-mix, wet-mix or both.
- Minimum of five (5) years experience presenting information regarding shotcrete applications in: education, marketing service or similar instructional presentations for shotcrete-related products or shotcrete equipment.
- Minimum of five (5) years experience in the design community directly involved with shotcrete specification, application, and quality control procedures.
- Minimum of five (5) years experience in shotcrete materials, equipment manufacturing, sales or service of the same application experience.
- Minimum of five (5) years experience in quality control or lab testing procedures related to shotcrete.
- Minimum of five (5) years teaching experience.
- Be a current ASA member (Individual member, primary contact of Corporate membership, or Corporate Additional member).
- Have a resume demonstrating basic shotcrete knowledge.

Upon probationary approval, the applicant must have participated in all phases of at least two (2) ASA-sanctioned Nozzleman Education programs for each process for which approval is sought, with different educators of record for each session. For the first session, the applicant must attend the session as a witness to the supplemental educator. For the second session, the applicant must attend the session as a supplemental educator and shall conduct all phases of the session under the direct supervision of the educator of record.

Unlike ACI certification, it should be noted that attending an ASA Education session does not require any pre-requisite, such as 500 hours of nozzling experience, for the education session to take place. Attending an ASA shotcrete nozzleman education session is a valuable tool for nozzlemen wishing to improve their knowledge and placement skills, and for others who want to learn more about the latest in shotcrete technology and equipment. This program is also a valuable tool for those interested in becoming involved in the shotcrete industry.

The goal of the Association is to promote shotcrete throughout North America. Further developments are to have ACI Local Chapters certification sessions on a regular basis covering various key locations. ASA has sponsored or overviewed over 400 shotcrete educational sessions to this day. The next task for the ASA Education Committee will

be to develop a Shotcrete Inspector's Education Program for Inspectors involved in the shotcrete industry.

2.2 Underground shotcrete education

In an effort to improve and ensure the quality of their construction projects, mine operators, tunnel owners and underground contractors expressed an interest in educating those members of their work force involved in the placement and inspection of shotcrete for ground support. A program providing the basics of the shotcrete process, including materials selection and testing, basic equipment requirements, skill requirements for the nozzlemen and crew, proper shotcreting techniques, and safety considerations in the underground environment was developed.

Educators are selected on the basis of their experience and knowledge of the use of shotcrete for ground support. They must meet a similar list of requirements (related to underground shotcrete), as the one described earlier.

2.3 ACI Shotcrete Nozzleman Certification program

In addition to the ASA shotcrete education program, the ACI Shotcrete Nozzleman Certification program provides stringent standards when it comes to shotcrete quality. In addition to good quality, well-proportioned shotcrete mixes and suitable well maintained equipment, qualified and certified shotcrete nozzlemen are a key element to the success of this process. The ACI, through its C660 Committee, is delivering a credible and thorough program with strict policies, guidelines and procedures that respond to the demands of the construction industry (Morgan & Dufour 2008).

The program is continually improving over time as active ACI shotcrete examiners are also members of the C660 Committee. Since nozzleman certification can be more hazardous than any other ACI programs, one of the documents the Committee has developed is a performance specification document. This document will assist hosts in setting up certification sessions in a safe manner, including guidelines for overhead panel affixation.

Overall, the number and location of ACI shotcrete nozzlemen certified throughout North America for both the dry and wet-mix processes since the start of the program until now is illustrated in Figures 1 and 2. Since 1904, ACI has been a leader in producing consensus committee documents for the concrete industry: codes, specifications, reports, and education materials. Since the beginning of the shotcrete nozzleman certification program, the ACI C660 Committee has continued

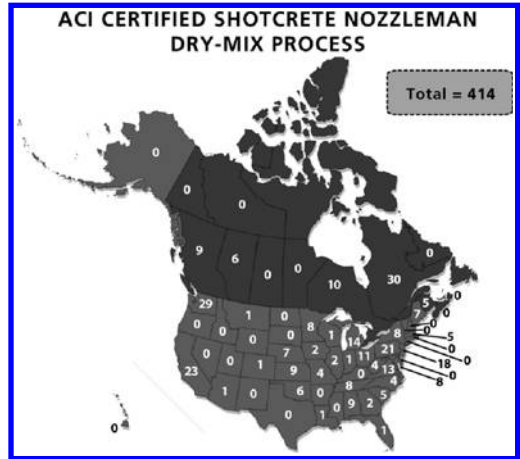


Figure 1. ACI Certified *dry-mix* Shotcrete Nozzlemen in North America (as of July 2009).

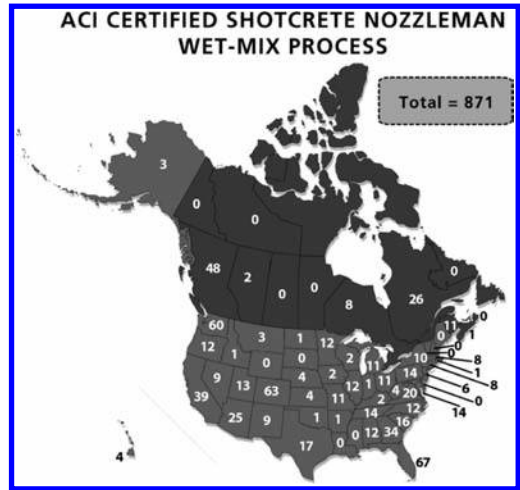


Figure 2. ACI Certified *wet-mix* Shotcrete Nozzlemen in North America (as of July 2009).

to improve the scope of the program and the quality of the study documents, making it more nozzleman user friendly.

2.4 CCS-4 (08) Shotcrete for the Craftsman

One of the education documents also used by nozzlemen for the certification program, *CCS-4 Shotcrete for the Craftsman*, has been remodeled and now includes an easy-to-read section on concrete technology. This section has been adopted to introduce the shotcrete process as a method of placing concrete. It was agreed by committee members that shotcrete craftsman should understand the basics of concrete production and behavior.

Shotcrete workers should also understand safety procedures, including the use of personal protective equipment when they are placing and finishing concrete. A series of detailed illustrations were added to this version of CCS-4 (08) Shotcrete for the Craftsman (2008).

Examples are presented in the following figures of illustrations incorporated into the new CCS-4 (08) Workbook:

The new Workbook document that was recently published in 2009 by ACI C660 is the CP-60 (09) Craftsman Workbook for ACI Shotcrete Nozzleman Certification (2009), is comprised of the following:

- Program Information,
- New CCS-4 (Shotcrete for the Craftsman, Concrete Craftsman Series 4),
- Appendices.

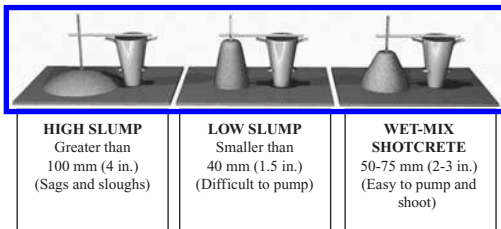


Figure 3. Measure of workability of wet-mix shotcrete.

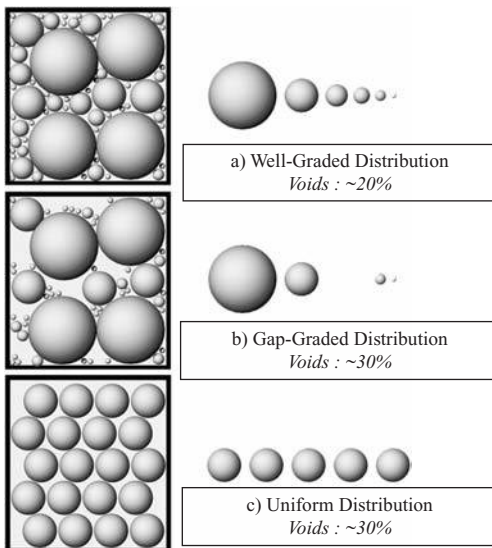


Figure 4. Illustration of different aggregate size distributions: a) well-graded distribution, b) gap-graded distribution, and c) uniform distribution.

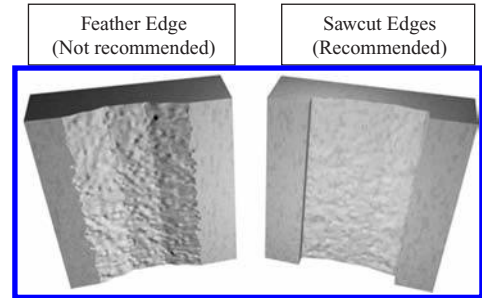


Figure 5. Concrete surface preparation—feathered vs. sawcut edges.

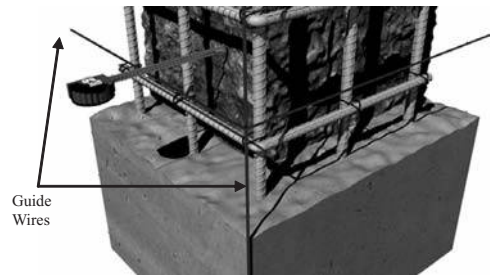


Figure 6. Guide wires installed vertically and horizontally. Wires strung along to the finished grade line provide a visual guide for the nozzleman and finishers.

The Committee is in the process of reviewing the written examination questions based on the new literature. This international certification program and its supporting literature (Workbooks, written examination, etc.) are also available in three languages: English, Spanish and French.

2.5 Shotcrete core grading

The use of shotcrete core grading to quantify the quality of shotcrete cores has been overused for years and, in the past, was included in many ACI shotcrete related documents. With the exception of the nozzleman certification program, core grading has been discontinued. Industry experts felt that it was risky to propose such a subjective tool to members of the engineering and construction industries with very little experience in the field of shotcrete (problem related to its misuse). The core grades example displayed in the appendix document did not necessarily represent nor cover all the structure configurations where shotcrete is applied, and therefore, made it irrelevant.

Core grading however is an appropriate means to evaluate the skills of nozzlemen with respect to

shotcrete consolidation and rebar encapsulation. Although it is still a subjective method, it allows consistent grading amongst ACI Examiners. The size (length and diameter) of the cores, the core locations (5) in the panel, the size and configuration of the reinforcing steel and the size of the panels are consistent parameters important for a standardized certification program (Figure 7).

It should be noted that a total of five cores are graded using these criteria at specific locations. A test panel with any single core grade exceeding core grade No. 3, or with more than two of the five cores having a core grade No. 3 is declared a failure. Averaging of core grades is not permitted. Definitions of core grades No. 1 to 5 are provided in the Program Policy (CP-60, 2009).

A new Task Group has been recently formed to evaluate and improve the current core grading appendix. While the current system only addresses rebar encapsulation, it often does not address any other defects within the cores. The objective is to describe potential defects and their position (adjacent or not to the rebar), including evidence and size of sand lenses, pockets and voids in order to provide better analysis in the examiner's reports to ACI.

As an example, a core grade #3 can be hard to evaluate and display different defect patterns (Figure 8). This new document will be used as guidance to improve accuracy and consistency

during the visual examination, and to help all ACI examiners coming from different locations throughout North America. This document is under review by ACI Committee C660 (as of July 2009).

2.6 Shotcrete inspector program

In parallel to the current ACI Shotcrete Nozzleman Certification program, ACI is looking into a special certification program aimed at professionals inspecting shotcrete work. Presented as the *Shotcrete Inspector Program* for the moment, early industry responses to the proposition are extremely positive. As mentioned earlier, the ASA is also working on developing education modules specific to professional shotcrete inspectors. Both these education (ASA) and certification (ACI) programs should be operational in a near future.

2.7 Shotcrete nozzleman education and certification around the world

Shotcrete education and nozzleman certification is not only limited to North America. A wet-mix shotcrete nozzleman certification session recently took

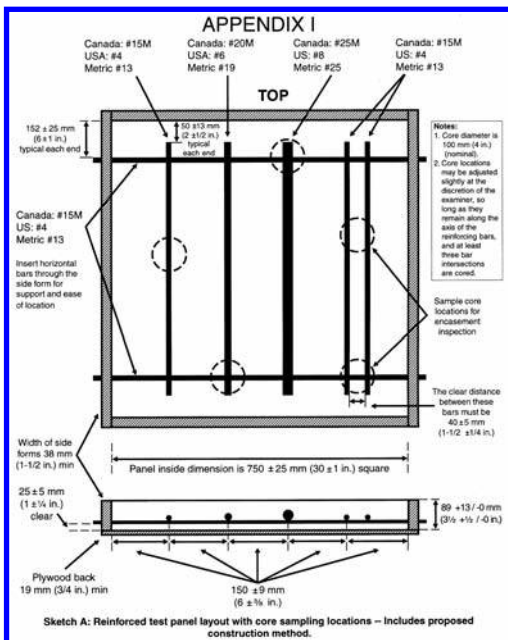


Figure 7. Standard ACI Shotcrete Nozzleman Certification Panel (2009).

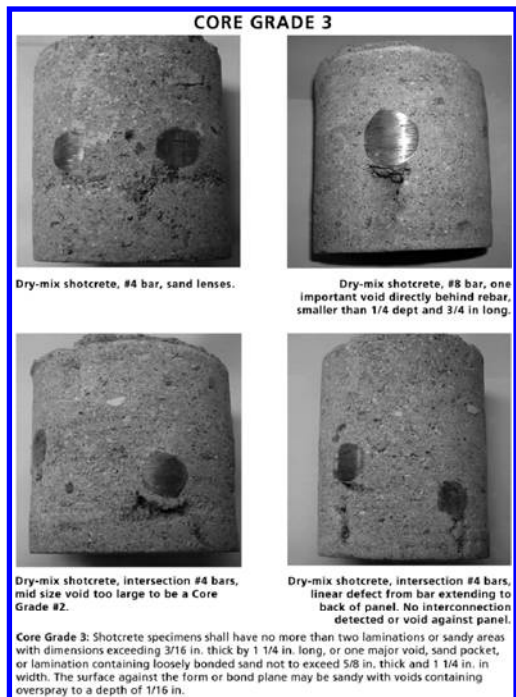


Figure 8. Suggested Core Grade Document—ACI Shotcrete Nozzleman Certification (Currently under review by ACI Committee C660 as of July 2009).

place in Singapore for more than 10 nozzlemen! A series of articles illustrating available nozzleman education and certification programs in Europe, South Africa, South America, and Australia have been published and presented at previous conferences (Larive 2009; Boniface 2008).

The quality of shotcrete is highly dependant on the skill of the Nozzleman. Excellent education and certification programs around the world are available to continually improve the quality of shotcrete placement leading to better quality work. Regardless of the industry type (civil, repair, structural work, tunneling, mining, pool and spa construction, etc), we shall all use these available tools to improve knowledge and awareness of the shotcrete process. As an industry, we have set the bar high, but collectively, we can raise it even higher!

REFERENCES

- Boniface, A., 2008. Some Aspects of International Shotcreting Practice, SANCOT Seminar: Ingula Pumped Storage Scheme, The Southern African Institute of Mining and Metallurgy, November 7–8.
- CCS-4, 2008. Shotcrete for the Craftsman, American Concrete Institute.
- CP-60, 2009. Craftsman Workbook for ACI Certification of Shotcrete Nozzleman.
- Dufour, J.-F., 2008. Shotcrete Nozzleman: ASA Trains—ACI Certifies, ASA Shotcrete Magazine, Volume 10, Number 1, Winter, pp. 8–12.
- Isaak, M., 2002. Certification vs. Qualification of Shotcrete Nozzlemen, ASA Shotcrete Magazine, Vol. 4, No. 4, Fall, pp. 40–42.
- Larive, C., 2009. Why and How to Certify Sprayed Concrete Nozzle Operators? Shotcrete for Africa Conference, March 2–3.
- Morgan, D.R., Dufour, J.-F., 2008. Shotcrete Nozzleman Certification and Qualification in North America, Modern Use of Wet-Mix Sprayed Concrete for Underground Support, Fifth International Symposium on Sprayed Concrete, Lillehammer, Norway, April 21–24, pp. 269–277.

Robotic shotcrete shaft lining—a new approach

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ABSTRACT: Macmahon has recently designed and constructed its first robotic shotcrete shaft liner (hereafter referred to as “shaft liner”). The shaft liner was engineered with two principles in mind: enhancing the safety of operators working around an open hole, and improving the quality of the final shotcrete lining. The result is a state-of-the-art machine which has raised the standard in the industry for operator safety, rebound reduction and quality of lining.

Design and construction of the shaft liner winch deck platform enables the unit to be set up in close proximity to the collar of the hole without exposing personnel to the associated hazards. The shaft liner incorporates scanning technology to enable both pre and post scanning of the shaft surface area to gauge an accurate thickness measurement of the applied shotcrete. Recorded video footage of the shaft is also captured separately on DVD to allow close inspection of the condition of shaft walls and identification of any faults or over-break. Additionally, shotcrete application is monitored and recorded to demonstrate spray consistency. This paper will discuss all aspects of the design, engineering, commissioning and operation of the robotic shotcrete shaft liner.

1 INTRODUCTION

Macmahon Mining Services is a division within Macmahon that offers specialised underground services to the Australian and International mining industries. The division encompasses ground support installation in the form of cable bolting and shotcreting, raise drilling and specialised engineering services including shaft fit-out. In order to be able to provide clients with a full suite of options for the development and support of vertical openings, a remote shotcrete shaft liner has been developed. After reviewing the existing shaft liners operating in the mining industry, Macmahon drew on this collected experience and designed and constructed a state of the art machine.

2 HISTORY OF DEVELOPMENT

Historically, installation of surface support in shafts has been accomplished through man access to the shaft via a hoist and stage enabling direct installation of shotcrete, mesh or concrete lining. Working in shafts is a high risk activity in mining and also adds significant cost to the development of a shaft due to the specialised equipment and personnel that are required. An alternative to manual application that may be viable in some circumstances is to use a machine to remotely apply shotcrete within the shaft.

There is a paucity of published material in the industry concerning shotcrete shaft lining, making it difficult to examine the evolution of techniques and current practices. Hustrulid & Bullock (2001) refer to “Shelob shaft shotcreting units” developed by Caledonian Mining Co Ltd, but no further detail is given. Rispin et al (2005) refer briefly to “shaft robo” machines used for application of shotcrete, although this type of machine appears to be designed to be slung beneath a man-access stage rather than to operate remotely. An ITA/ATIA report in 2006 entitled “Shotcrete for Rock Support—A Summary of the State of the Art” refers to a submission from Canada detailing the “completely robotic, continuous placement of 75 mm of shotcrete in a 415 metre deep, 2.4 metre diameter shaft using wet mix materials and placement” but does not give further details.

An examination of websites from companies offering shotcrete shaft lining as a service perhaps gives a better idea of the technology on hand. These companies include Jetcrete Australia, Rix Group Australia and Multicrete systems in Canada. Jetcrete operates three shaft lining machines and the basic operation of their machine is described on their website. Rix group is another Australian company with remote shaft lining capability. Multicrete systems have developed a “Raise Robot” shotcrete machine and a basic description of its operation can be found on their website. Macmahon has drawn on the experience of all of these

machines to design a shaft liner with a similar, but refined mode of operation to the above.

3 DESIGN CONSIDERATIONS

3.1 General

Raise boring of shafts has become a popular method of establishing vertical and sub-vertical openings in both mining and civil applications. In general, openings that are raise-bored are suited to being lined using a remote shotcrete shaft liner. Raise boring is only undertaken in competent ground due to the stand-up time required for the walls of the shaft. This usually means that the shaft will be competent enough to support with shotcrete alone and that it will remain open long enough to enable application of this shotcrete using a shaft liner. The use of a raise drill in combination with a remotely applied shotcrete lining reduces costs significantly.

Raise bored shafts are constructed both from a pre-existing underground opening to the surface or between underground openings. Thus the shaft liner had to be designed to either operate on the surface or from a shaft collar in a chamber underground. This requirement restricted the potential dimensions of the shaft liner.

There are two commonly used shotcrete application methods: the use of dry mix shotcrete where water is added to the dry materials at the nozzle during spraying and the use of wet mix where the water is mixed into the concrete at batching. Dry mix offers significant advantages for a remote shaft lining application such as unlimited holding time for dry materials (provided they have not been pre-dampened), long conveying distances possible and lower weight of material in lines down the shaft. Wet-mix also tends to segregate when used in these types of applications. Therefore a dry mix process was selected for use with the shaft liner.

3.2 Hazard management

Conducting shaft lining activities exposes personnel to several major hazards that need to be managed. The hazards which result in the greatest risk to personnel are working around open holes and working around moving lines. Working around an open shaft collar is necessitated both during the setup of the machine platform, the lowering of the robot into the hole and the operation of the machine when lengths of hose are being added. Both physical barriers and safe work procedures are necessary to mitigate the risk of falling.

During the lowering of the robot down into the shaft there is the requirement for feeding lengths of materials hose, water hose and communication

and power cables over the collar and into the hole. Personnel are required to work in the same area as these moving lines, creating a hazard of interaction which could result in persons being dragged towards an open hole.

3.3 Quality of application

As with any shotcrete process, the quality of the final lining is dependent on the application method. The use of the dry mix method necessitates careful control of rebound and the water content of the mix. Controlling the water content of the applied shotcrete can be difficult as matching of the shotcrete machine speed and the water flow rate at the nozzle is required. Too little water can result in poor mixing and inadequate hydration of the cement, while too much water will lower the strength of the material.

The performance of any shotcrete lining is controlled by the thickness at which it is applied. The ability to accurately measure the final lining thickness is essential to ensure the quality of the application.

4 FINAL DESIGN

Following extensive research, construction of the shaft liner commenced in 2008 and it was completed and commissioned in January 2009. Research involved a comprehensive risk analysis, assessment of existing equipment within the industry and discussions with various shotcrete equipment manufacturers.

4.1 Shaft liner elements

The shaft liner consists of four main elements: the control room, the dry-mix shotcrete machine, the robot which is lowered down the shaft and the winch deck platform which houses the winch, boom, hose reels and associated equipment.

The control room is housed in a 3 m sea container and contains the operator control and monitoring system. The shaft liner operator can control all functions of the robot operation from this room including lowering and raising of the robot, rotation of the nozzle, water metering, spraying and scanning. The shotcrete machine used is a Reed Series 5 machine. The machine is typically fed directly by a concrete agitator truck but could also be loaded using 'bulk-bags'. A machine operator is required to control the filling of the machine throughout operation.

The basic setup of the platform and robot is illustrated in Figure 1. The robot has three legs the length of which can be adapted to fit the diameter of the shaft that is being sprayed. The nozzle

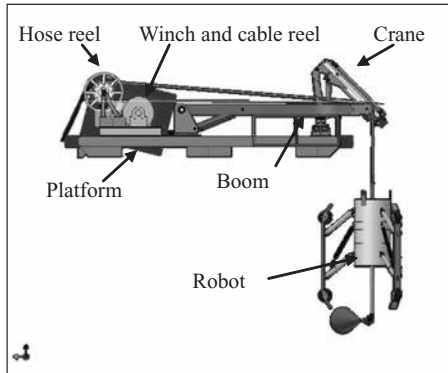


Figure 1. Setup of platform and robot.

is suspended below the main body of the robot and can rotate through 360 degrees. The robot is also fitted with cameras to monitor the spraying.

The boom of the platform can be extended, raised and lowered as required to position the robot within the hole. The hoses and cables run along the top of the boom and are contained within a covered channel. The winch and cable reel control the lowering and raising of the robot within the shaft. The water hose and power cables are stored on a hose reel, while the concrete hose is fed through from the rear of the platform. A crane is mounted at the front of the boom to enable the robot to be lifted into and out of the shaft collar.

4.2 Rig specifications

The remote shaft liner has been designed to be able to operate either on the surface or in an underground chamber. The platform is 2 m in width, 6 m in length and has a maximum height of 2 m enabling it to be situated in most standard underground drives. The control cabin is housed in a sea container which is easily transported underground. The shaft dimensions that the shaft liner is able to operate in are:

- Shaft diameter from 1.8 m to 8 m,
- Shaft inclination from vertical to 50 degrees
- Shaft depth to 350 m

An air volume of 25 m³/min at a pressure of 0.7 to 0.8 MPa is required for effective operation. The liner can be run on either 415 V or 1000 V power and sprays a nominal 10 mm dry mix shotcrete product. Shotcrete at a thickness between 10 mm and 100 mm can be applied in one pass.

4.3 Safety enhancements

A comprehensive risk assessment was conducted as part of the design of the shaft liner which has

led to several innovations in the management of the hazards present.

4.3.1 Open hole hazard management

The platform of the shaft liner has been designed with a boom and a crane to ensure personnel are never required to work within 1.5 m of the hole collar during normal operation. The platform is secured to the ground using “gewi” type resin bolts. The crane can lift the robot into and out of the hole collar and the boom can be used to extend out over the hole as required. This functionality means that the platform can be located back from the collar of the hole, not over the top of the collar as with some shaft liners currently in operation.

As there is no requirement to be within the immediate vicinity of the open hole, effective guarding can be in place throughout the operation (Figure 2).

Maintenance and repairs on the robot can be carried out well away from the open hole with this platform setup. Other shaft liners in operation may require work to be performed on the robot while it is held by the winch in the collar of the hole with personnel wearing fall arrest gear to manage the hazard. An elevated anchor point is located at the centre of the platform to allow fall arrest or restraint devices to be anchored if work is required closer to the collar of the hole for any reason.

4.3.2 Moving lines hazard management

The hazard of working around moving lines has been reduced significantly in the design of the shaft liner. The hoses and cables run through a fully enclosed channel on the top of the boom (Figure 3). The water hose is housed on a reel at the rear of the platform, while the materials hose is fed through a set of rollers at the rear of the platform. The power cables are secured to the water hose and stored on the reel also.



Figure 2. Guarding around hole collar.

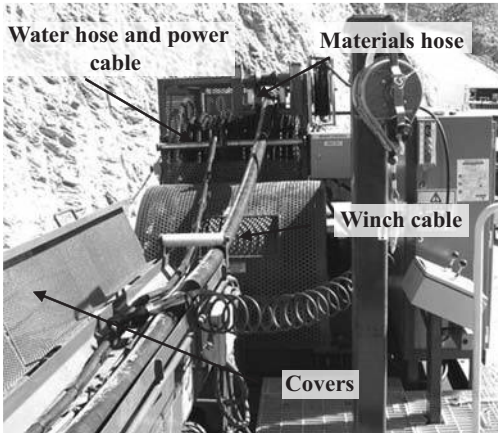


Figure 3. Enclosure of moving lines.

All hose reels are encased with wire mesh to prevent inadvertent access. Having the moving hoses and cables secured behind guarding also reduces the potential hazard presented by a broken hose as the free ends of the hose will be contained and unable to whip around.

Lengths of hose need only be added to the concrete line during operation. This operation is performed while the machine is shut down and hence no lines are moving while the operator clamps the hose on or off. As a safety precaution, an emergency pull stop cable has been installed on the platform which will shutdown all rig functions if activated in the event of an emergency.

4.3.3 Other safety features

The shaft liner has numerous other safety features including the shotcrete machine and winch deck being remotely controlled from the operator's cabin, removing the need for personnel to be in the vicinity of moving parts. Pressure gauges are located throughout the system and relay information through the Citect control system to the display in the operator's cabin. This information can be used to prevent over stressing of any part of the system.

A three point braking system has been fitted to the winch system to control rate of descent. To improve the ability of a single operator to control the entire process a moveable camera mounted on a pole relays images to the operator's cabin and can be used to monitor any area required. The robot is also fitted with methane sensors for working in underground coal mines. Laser barriers can be erected if required in access drives underground which will shutdown the rig if breached. These can be used to prevent inadvertent access to the bottom of the hole during spraying operations.

4.4 Performance and quality

The shaft liner was designed to offer benefits to clients in terms of its performance on site and the quality of the final lining achieved.

4.4.1 Performance considerations

Performance has been considered in terms of independence of operation on site, control of spraying process and efficiency of application.

Independent operation is beneficial for clients to minimise disruption to mine production. Minimal use of cranes is required with the setup of the shaft liner, with the only lifting required being that to remove the platform and sea containers from the truck and into position over the hole. The robot can then be correctly positioned using the lifting crane and adjustment using the boom. Control of the spraying operation is facilitated using a Citect control system and a graphic display in the control room (Figure 4).

The Citect system monitors all aspects of the spraying operation including depth of the robot, (which is measured using cable counters on the sheave wheel), nozzle rotation (which is measured using a proximity switch), winch speed, oil temperature, shotcrete machine speed, air pressure, water flow rate and any faults that occur. This system provides precise, real time feedback to the operator on all aspects of the spraying operation. Fault diagnosis is also provided by the Citect system enabling identification of the issue by the operator and promoting quick rectification with minimal downtime. Fully automated spraying is possible using this system.

There are four infra-red cameras (Figure 5) mounted on the robot facing the shaft walls at 90 degrees to each other. The use of infra red cameras means that only low light levels are required. Light is supplied using several LED lights mounted at the top of the robot. Infra-red cameras have the



Figure 4. Graphic display in operator cabin.



Figure 5. Infra red camera mounted on robot.

benefit of providing a clear picture to the operator in all working conditions as the picture is less affected by dust and water vapour. The cameras are mounted within an enclosed housing for protection and are kept free of dust by air and water sprays.

Having a clear view of the spraying is vital for the operator to assess the consistency of the mix being attained, the amount of rebound that is occurring and where any variances in shaft wall conditions may necessitate further spraying. The footage from the cameras is recorded and can be copied to a disc and given to clients for review if required.

4.4.2 *Quality of final lining*

The quality of the final applied lining is, of course, the most important performance consideration. As with any dry mix process, the control of accelerator and water dosing is critical to the quality of the product. The thickness of the lining obtained is the other major factor affecting the behavior of the lining.

Water metering is finely controlled to an accuracy of 0.05 L/min by the operator via the Citect system in the control room. The water meter is mounted on the robot as opposed to being on the water line at the surface meaning that the true water being delivered to the nozzle is measured. Liquid accelerator is used in place of the powdered accelerators traditionally used in this type of application. Liquid accelerator is added at the nozzle whereas powder accelerator is added at the shotcrete machine with the dry mix. The advantage of using liquid accelerator is that an even dose rate of accelerator can be achieved.

The shaft liner is equipped with a thickness scanning system using an ultrasonic distance measurement device. The scanner is set up to measure 20 points at each depth with the nozzle rotation and depth being indexed against each measurement. A scan of the entire shaft is conducted prior to spraying and a second scan is conducted post spraying. The difference in distance measurement at each point can then be calculated to give a thick-

ness of lining and indicate any areas of possible under-spray that require additional spraying. The rate of rotation of the robot must be determined at the start of spraying to enable the desired thickness of lining to be achieved. This is done by spraying a small section of the shaft, scanning that section and adjusting the required parameters to suit.

5 COMMISSIONING

Commissioning of the shaft liner was carried out in January 2009 on a 3.5 m diameter shaft designed to be lined to a depth of 74 m at a thickness of 50 mm. The shaft collar was located in the floor of an open pit. The commissioning was used to calibrate the various measuring devices on the shaft liner, though several other issues emerged with the operation of the shaft liner during this time. These included:

- Nozzle getting caught up during rotation
- There were originally no lights installed on the robot, resulting in the infra-red cameras having difficulty in judging the depth of field for focus
- There was some brake chattering evident.

The nozzle rotation issue was solved by replacing the swivel at the top of the robot with an alternative type that allowed free rotation. With the installation of the LED lights on the robot, the cameras were able to judge depth of field correctly and focus appropriately providing clear images to the operator. The brake chattering issue was overcome through circuit changes.

The commissioning of the shaft liner was completed in seven days with a total of 43.5 m³ of fibre reinforced shotcrete being sprayed in the shaft. The shaft liner performed well, achieving a consistent and smooth lining with minimal rebound. Figure 6 shows the shaft liner robot suspended in the shaft during commissioning.



Figure 6. Shaft liner robot suspended in shaft during commissioning.

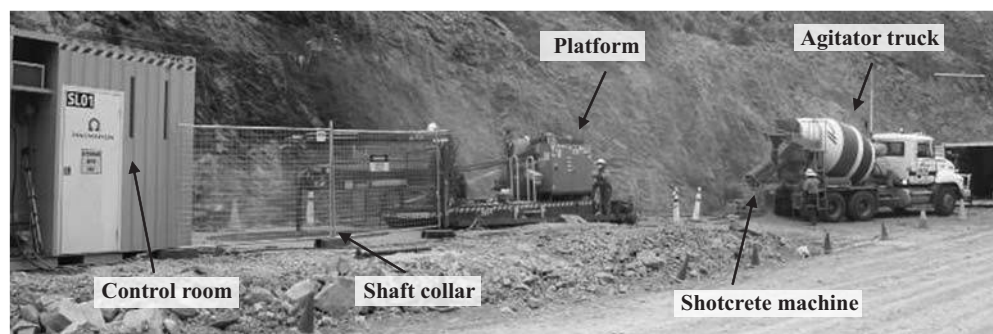


Figure 7. Shaft liner setup during commissioning showing major components.

6 PERFORMANCE TO DATE

To date four shafts have been lined using the robotic shotcrete shaft liner. Shaft diameters between 3.1 m and 5.4 m have been successfully lined with the maximum design depth of lining required being 185 m. Lining thicknesses applied to date have been either 50 mm or 75 mm nominal thickness.

From this experience it has been found that application rates of up to 16 m³ in a 12 hour shift can be achieved with an overall average of approximately 13 m³/shift. To give an idea of the rate of coverage that the maximum application rate equates to, for a 50 mm lining applied to a 5 m diameter shaft with 20% rebound allowance, 17 m of depth in the shaft would be lined per shift.

Rebound varies with the concrete mix supplied, but is within the range of 10–20% which is exceedingly good for dry-mix shotcrete. The reason for this low rebound is thought to be the effect of the high degree of control of water and accelerator dosage during spraying. The aggregate type and grading is the factor found to have the most effect on the variance in rebound experienced.

7 FUTURE WORK

The robotic shotcrete shaft liner developed by Macmahon represents a new approach in terms of safety and quality assurance to remote lining of vertical and inclined openings with shotcrete. Future development of the shaft liner is planned to increase its application and refine the control systems.

The current depth that the shaft liner is thought to operate effectively at is 350 m. This is due to the amount of air volume that can be supplied to keep the dry mix moving in the lines and prevent compaction of the mix and hence blockages at the robot. This maximum depth restriction will be investigated by Macmahon as shaft lining oppor-

tunities arise and further work is undertaken in an attempt to achieve operation at greater depth. The braking system is planned to be improved by adding a dynamic brake to operate along with the three point braking system. This enhancement will provide additional redundancy in the braking system and also facilitate greater control of the rate of descent at depth.

Further work is also planned on the scanning system to translate point data into three dimensional images and point data files that may be imported into mine survey software for analysis. Further feedback on shaft geometry will be provided by the installation of pneumatically controlled legs on the robot. These legs will adjust to the contours of the shaft and relay changes in the diameter of the shaft back to the operator as the robot travels in the shaft.

Macmahon anticipates that the safety features and performance of this shaft liner will help to raise the standards in the industry for remote shotcrete lining of shafts.

REFERENCES

- Hustrulid, W.A., & Bullock, R.C., Editors, 2001 *Underground Mining Methods: Engineering Fundamentals and International Case Studies*. Society for Mining Metallurgy and Exploration, Littleton, Colorado, 728 p., www.smenet.org
- Rispin, M., Gause, C., & Kurth, T. 2005. "Robotic Shotcrete Applications for Mining and Tunneling". *Shotcrete* Vol. 7, No. 3 Summer 2005, pp. 4–9.
- Shotcrete for rock support—A summary report on state-of-the-art*. 2006. ITA-AITES. www.ita-aites.org/cms.
- www.jetcrete.com.au/Services/Shaft-Lining-with-Shotcrete.aspx
- www.multicretesystems.com/images/Equipment%20-%20Robotic/Raise%20Robot.pdf
- www.therixgroup.com.au/

Advances in shotcrete impact-echo testing

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ABSTRACT: Impact-echo is a well-known non-destructive technique for in-situ evaluation of structural concrete members, progressively developed over the last 25 years. This study outlines recent technical advances in the application of impact-echo testing to shotcrete tunnel linings, for the purpose of measuring shotcrete thickness and providing physical properties related to substrate bond conditions and early-age strength. The impact-echo method is based on the analysis of a high-frequency resonance, and used to determine concrete thickness, integrity, and support conditions. Testing is performed by exciting a stationary mode using a light steel impactor, and subsequent data acquisition and analysis. Layer thickness is determined as a function of the resonance frequency, while attenuation characteristics relate to the support or bonding conditions at the opposite surface. In this case additional physical measurements of P-wave velocity and surface hardness are incorporated into the testing procedure, with the aim of providing an estimate of early-age strength. Specific challenges related to underground conditions, such as surface roughness and high operating frequencies are also addressed.

1 INTRODUCTION

The use of a shotcrete lining as a structural element for ground support is increasingly widespread in both underground mining and transportation tunnelling operations. In such applications the shotcrete layer, typically reinforced with fibres or steel mesh, acts in conjunction with rock bolts to distribute stress, support the rock face and prevent localized or general collapse. The shotcrete liner is usually designed and specified for construction on the basis of a minimum cover thickness, necessary to ensure adequate structural performance, toughness, and protect steel reinforcement from corrosion.

The nature of underground tunnelling operations incurs a significant demand for quality assurance due to the high potential human and financial cost of a structural collapse in this environment. A variety of existing and emerging procedures facilitate thickness control during spraying operations, however there remain few options, bar drilling or coring, to verify in-situ thickness information after the fact. This is relevant to existing tunnels where insufficient historical information is available, and where independent auditing of minimum cover for contractual and safety compliance purposes is required.

In addition to cover thickness, stability of the liner is contingent on material properties and the quality of bonding at the shotcrete/rock interface.

The quality of this bond is subject to deterioration over time, as the structure is subjected to stresses from seismic events, blasting and other routine mining operations. Delineation of potential flaws of this nature necessarily requires a form of mechanical testing, such as outlined in this study.

A third area of critical importance is the establishment or permissible re-entry time after shotcrete spraying, contingent on the monitoring of in-situ early strength gain of the fresh shotcrete.

This paper introduces the conceptual application of non-destructive testing (NDT) procedures specifically adapted for use on shotcrete linings. Applications include minimum thickness measurement, early strength estimation, and assessment of substrate bonding conditions using direct mechanical measurements. Impact-echo testing and related in-situ methods are proposed as a complement rather than a substitute for other direct measures that constitute the basis for existing shotcrete quality assurance programs.

2 INSTRUMENTATION

Unless otherwise stated, the data presented in this paper were obtained using a prototype shotcrete impact-echo system designed for operation on unfinished surfaces, high operating frequency range and optimized signal/noise ratio.

The instrumentation consists of a high-frequency instrumented impactor source, and 3 dry contact piezoelectric transducers mounted on a handheld frame (Figure 1). Data is acquired using an 8 bit digitizer, sampling at 0.2 microseconds per channel, and recorded to a PC for further analysis.

Innovative features of this equipment include the incorporation of an instrumented high-frequency mechanical impact source. This facilitates triggering at the precise excitation time in underground environments, where background noise from drilling and vehicle operations can be significant, and allows characteristics of the impact (namely contact duration and force history) to be recorded for further analysis. The impactor is mechanically driven, favouring both acquisition speed and repeatability of testing results.

The sensor system comprises three independently mounted displacement transducers. The sensors are spring mounted to provide constant contact pressure and facilitate the use of multiple transducers on a shotcrete surface, which in contrast to concrete is typically unfinished and presents a highly uneven surface, as shown in Figure 1. The sensor array system is used to enhance signal to noise ratio and the sensitivity to non-propagating modes.

The prototype shotcrete apparatus differs from traditional impact-echo systems in the acoustic properties of the source, in this case capable of excitation and measurement over a higher frequency range (20 to 50 kHz). This is due to the requirement to characterize thicknesses below 50 mm, whereas most conventional impact-echo systems are optimized for the 150–300 mm range. The operational range of the instrument can be adjusted by modifying the impactor characteristics prior to testing.



Figure 1. Prototype shotcrete impact-echo system undergoing field trials. The testing device comprises an automated impactor and three spring-mounted transducers, which are connected to a data acquisition module and computer, on the left.

3 IMPACT-ECHO THICKNESS

3.1 Background

Impact-echo is a proven and effective NDT method that has been used for the evaluation of in-situ concrete members such as slabs, pavements and girders for the past 25 years (Sansalone & Streett 1997).

The method is based on the analysis of a high-frequency acoustic resonance mode, which is excited by tapping the structure with a light-weight tuned steel impactor. It should be noted that upper and lower bounds of thickness resolution exist for any given testing configuration, the principal limiting factor being the harmonic properties of the impactor source.

Traditionally, resonance frequency has been associated with the return time of successive P-wave reflections between two adjacent surfaces. More recently, the impact-echo frequency has been identified as the S_1 Lamb wave frequency under resonance conditions, i.e. zero group velocity (Gibson & Popovics 2005). In either case, the measured thickness frequency (F) is defined in terms of material properties and section geometry:

$$F_t = \frac{\beta \cdot C_p}{2t} \quad (1)$$

where C_p is P-wave velocity, t is section thickness, and β ranges between values of 0.945 to 0.958 for normal concrete and shotcrete material properties.

The correction factor β is the ratio between the zero-wavenumber and zero-group velocity frequencies of the dispersive S_1 mode, as illustrated in Figure 2. The former is purely compressional but has negative group velocity, while the latter represents the actual stationary resonance condition. The correction factor is therefore necessary to adapt the measured stationary resonance to an interpretation based on C_p alone (Gibson & Popovics 2005).

According to the standard impact-echo procedure (ASTM 2004) the P-wave propagation velocity for the material must be determined a-priori for calibration purposes. In practice this calibration may be carried out by one of the following methods, listed here in decreasing order of precision:

- Performing a resonance test on a known thickness of representative material,
- Measuring surface-skimming Rayleigh-wave arrival at increased source-receiver offset, and the application of appropriate corrections (Popovics et al. 2006),
- Measuring surface-skimming P-wave arrival at increased source-receiver offset.

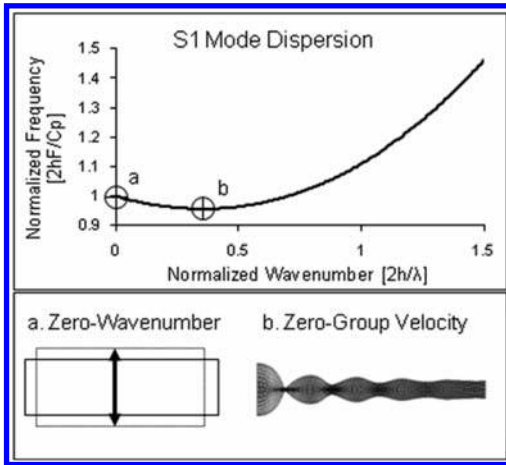


Figure 2. Distinction between zero-wavenumber and zero-group velocity frequencies, as defined by the analytical S1 mode dispersion curve. The lower sketches indicate exaggerated deformation patterns associated with each S1 wavenumber.

3.2 Example data

The form of data obtained using the prototype shotcrete instrument on a free plate is shown in Figure 3. This includes impact data, a processed transient response and a real-time thickness plot, obtained by applying Equation 1 to the FFT frequency spectrum of the windowed and filtered response, assuming prior calibration using one of the above-mentioned methods.

The accuracy of impact-echo thickness measurement is contingent on the quality of the C_p value used, and is therefore affected by inhomogeneities in the material as well as the calibration technique. Given the availability of high-quality calibration data, thickness accuracies to within $\pm 3\%$ have been attainable when testing concrete slabs (Gibson 2005).

Impact-echo is limited to the measurement of a single thickness layer, and results are affected by internal flaws that manifest a reduction in cross-sectional stiffness, such as corrosion-induced cracking and voids. Mechanical discontinuities presented by complete delaminations or poorly bonded cold-joints will result in measurement of the debonded section thickness only.

The minimum measurable thickness remains a function of the harmonic properties of the source, and the linear range to the receivers. Shallow delaminations and unbounded thin sections give rise to the excitation of a distinct low-frequency flexural resonance mode that is audible to the human ear, and the basis for the acoustic sounding method.

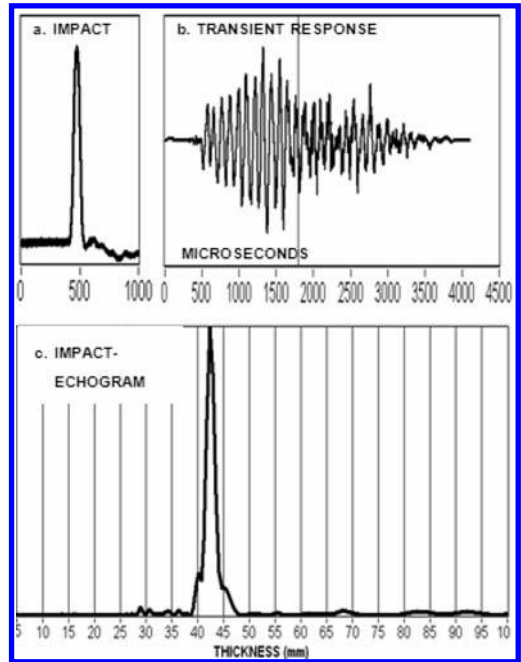


Figure 3. Sample impact-echo data as displayed in real time on the prototype shotcrete testing system: a) shows the impact signal and verification of valid trigger sequence, b) shows the windowed and band-pass filtered time-domain data, c) impact-echo thickness plot, given prior calibration to known P-wave velocity.

4 BONDING CONDITION

4.1 Background

In addition to thickness measurements, impact-echo has been used in the past to evaluate support conditions on a relative scale for the purpose of identifying subgrade voids, grouting integrity or the bond quality of shotcrete to a rigid substrate.

Such analysis is based on the effect of contrasts in acoustic impedance at the interface between concrete or shotcrete and the substrate material. This contrast affects the amount of vibration energy leaking into the substrate, therefore influencing attenuation of the impact-echo resonance. When the contrast is large, such as a free plate or a mechanically de-bonded shotcrete section, the rate of attenuation is lower—i.e. the section ‘rings’ for a longer period of time. As the impedance contrast decreases the impact-echo resonance attenuates more rapidly, until a practical limit of insufficient contrast, where impact-echo thickness testing would be unattainable. This differs from acoustic sounding in that it is based on the analysis of high-frequency cross sectional modes, as well as flexural

modes, and allows qualitative assessment of data at any location rather than a subjective pass/fail judgement of severe delaminations.

In practical terms, the contrast governing attenuation behaviour is a function of the relative material properties and the mechanical quality of bond at the interface. The observed effect on attenuation has given rise to several recent studies on time-frequency analysis of impact-echo data. One such case features experimental data from shotcrete samples overlaying granite substrate with simulated debonding flaws. This study demonstrates the value of attenuation analysis in underground tunnelling operations, while also showing thickness measurement to be still attainable for a fully bonded shotcrete/rock interface (Song & Cho 2009).

Time-frequency analysis has also been used to detect grouting flaws behind prefabricated concrete segments in shield tunnelling (Aggelis et al. 2008), as well as internal damage in concrete bridge decks (Shokouhi et al. 2006).

4.2 Example data

Experimental data presented herein to illustrate the concept were obtained from tests on a 180 mm thick, 2-by-2 m concrete slab placed over freshly mixed lean mortar, simulating the interaction of an increasingly rigid substrate bond as the mortar set and cured (Gibson 2005). The substrate mortar mix consisted of a 6:1 mass proportion of sand to Type I Portland cement mix, and sufficient water to provide adequate consistency and bonding to the pre-existing slab. The fresh mortar was placed in a wooden frame to a depth of 75 mm, and levelled prior to lowering the concrete slab over the top.

Impact-echo resonance measurements were performed at 1, 4, 8, 12, 22, 30 and 48 hours after the fresh subgrade material was first mixed. The data acquisition setup consists of 2048 sampling points at an interval of 5 μ s, using a conventional low-frequency, single channel impact-echo system. The source used for this data was a manually operated steel wire and ball-bearing impactor, tuned to excite the resonance frequency of the slab, approximately 11,200 Hz.

Impact-echo data was collected at 8 marked points on the top surface of the slab, each test repeated 5 times. In the first instance, the typical time-domain transient response at a given location, after 1 and 48 hours of subgrade curing time, is compared in Figure 4. Because the material properties of the slab are constant, any variations in its dynamic behaviour reflect physical changes at the subgrade interface. The effect of leakage attenuation as the subgrade material hardens is apparent as a dampening effect on the

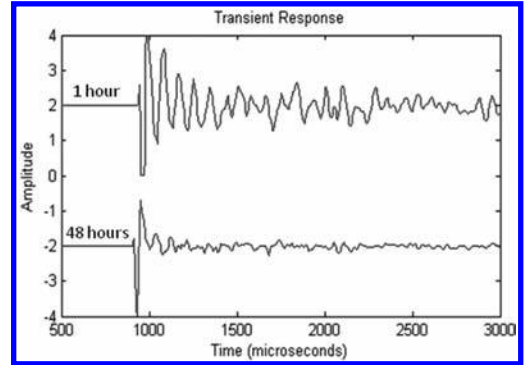


Figure 4. Transient response of concrete slab on a fresh mortar substrate, 1 and 48 hours into curing.

impact-echo resonance. A high-amplitude pulse at the beginning of the active response, corresponding to Rayleigh wave arrival, is not attenuated to the same degree as high-frequency components of the Rayleigh wave do not penetrate the full depth of the slab.

While contrasts in the transient response characteristics are discernible in light of control data, the concept of time-frequency analysis has been proposed by others to provide a more robust field method. This process should permit the assessment of data according to conformance with pre-defined benchmarks, without a requirement for control data.

Time-frequency data for this test was evaluated using a short-time Fourier transform, with 200 5 μ s samples per segment. Each data segment was windowed and zero-padded to 1024 points. The sampling window was subsequently advanced by 25 μ s per step, providing a high degree of overlap manifest in the smoothing of the contour plots. Examples of this form of data are shown in Figure 5. At 1 hour curing time the system models an un-bonded resting on grade. As noted elsewhere (Song & Cho 2009), the prevailing frequency band for this condition is narrower, and as bonding or substrate compliance increases the response is shorter in time and more distended along the frequency axis.

As a standard, objective methodology we propose to plot maximum time-frequency values along the time axis, defined here as 'peak spectral density'. In this case the data is normalized to maximum Rayleigh wave amplitude, to compensate for variations in energy imparted by the manual impactor.

Results from this analysis show a consistent pattern of reduction in peak spectral density as the grout below the slab cures, resulting in increased subgrade stiffness and the development of an

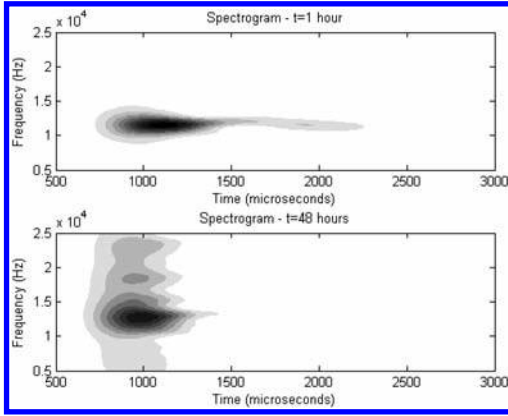


Figure 5. Spectrogram of concrete slab on a fresh mortar substrate, 1 and 48 hours into substrate curing. Average of 5 measurements at a single location.

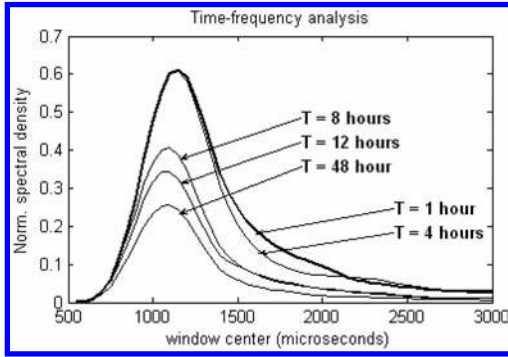


Figure 6. Time-frequency analysis: Peak spectral density between 5–25 kHz, normalized to peak Rayleigh wave amplitude. Average of 40 tests over 8 locations.

interface bond, and higher attenuation of the full-section resonance mode. Average data from all 40 tests (8 locations, 5 repetitions) are plotted in Figure 6, showing contrasts between 1, 4, 8, 12 and 48 hours of substrate curing time.

5 EARLY AGE STRENGTH ESTIMATE

5.1 Background

Shotcrete compressive and shear strength are engineering properties highly relevant to the structural performance of the ground support system. Of particular interest is the development of early-age strength, as this will often dictate the permissible rate of construction operations, as well as identify potential placement and curing problems.

In underground mining operations, in-situ early age strength estimation is pivotal to the determination of safe re-entry times (Bernard 2008).

Strength properties are typically measured directly, by testing samples of the material to destruction, such as cylinder compression tests and flexural toughness testing on FRC panels (ASTM 2005). In-situ, non or partially destructive methods provide complementary means to measure relative strength. These are based on physical measurements that relate to strength, usually via a test-specific empirical calibration. Although these methods cannot be as precise as destructive testing, they are invaluable for the purpose of verifying in-situ placement and curing, and providing an estimate of strength in situations where samples do not exist.

The following section outlines two physical measurements that are related to in-situ strength, and are obtained as a ‘by-product’ of the impact-echo procedures outlined above; therefore they provide complementary information that is of great potential benefit to the user.

5.2 P-wave velocity

P-wave or Ultrasonic Pulse Velocity is a long established technique to evaluate concrete quality, including delineation of internal flaws, micro cracking and estimates of compressive strength. The P-wave propagation velocity (C_p) in a homogeneous elastic medium is defined in terms of material properties as follows:

$$C_p = \sqrt{\frac{E(1-\nu)}{\rho(1+\nu)(1-2\nu)}} \quad (2)$$

where E is low-strain the elastic modulus, ρ is density and ν is Poisson's ratio.

The operating frequencies of the impact echo test range from 5 to 50 kHz, limiting the minimum wavelength to approximately 7 cm. As such, the method does not have sufficient resolution to discriminate material inhomogeneities such as cement, aggregate, fibres and mesh sections.

Although there is a correlation between pulse velocity and strength, this relationship is affected by other factors such as water/cement ratio and cement and aggregate types, therefore any attempt to apply a global conversion factor would be unreliable and is not recommended (ACI-228 2003). There is, however, an abundance of research showing mix-specific correlations between strength and C_p , that may be empirically established through laboratory testing (Pessiki & Johnson 1996) and may take the form of an exponential curve shown in Figure 7 (Lee 2003). On this basis, we propose

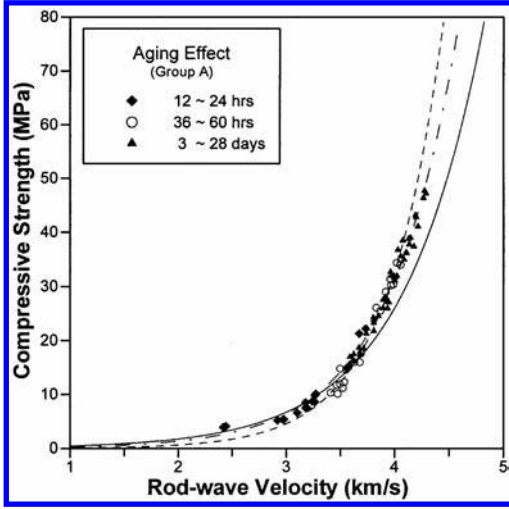


Figure 7. Example of a mix-specific, time-dependent P-wave velocity vs. compressive strength calibration for concrete (Lee 2003).

the incorporation of in-situ P-wave velocity measurements as a complement to early-age strength monitoring programs, in conjunction with direct measurements and the use of predefined, mix-specific calibrations.

5.3 Dynamic hardness

A second common approach to testing of concrete and other materials is through impact and dynamic hardness measurements. The analysis of impacts to characterise material properties is a widely studied subject, dating back over 100 years (Tiemann 1909). A practical example of this principle is the rebound or Schmidt hammer, a well-established technique for concrete and rock hardness testing (ASTM 2008), although of limited practical application to shotcrete. Implementation of the rebound hammer method to estimate compressive strength is based on empirical calibrations, and accuracy of inferring compressive strength from in-situ dynamic hardness using the Schmidt hammer has been estimated to be 15–20%, provided mix design and curing specific calibration curves are available (Malhorta & Carino 2004).

In the following analysis we examine the potential to utilize recorded impact characteristics for estimation of in-situ material properties. One of the innovative features of the prototype instrument is the use of an instrumented impactor, capable of measuring relative impact force history. Given consistent impactor excitation velocity, the contact duration (T_c) and maximum impact force (P_{cm}) are identifiable parameters that are sensitive to the

material properties of the surface. The procedure adopted to extract these parameters from experimental data was to fit a Gaussian approximation to the contact force history ($P_c(t)$), of the form:

$$P_c(t) = P_{cm} e^{-\frac{t^2}{(T_c/2)^2}} \quad (3)$$

Such an approximation and experimental data corresponding to impacts on concrete and granite samples are compared in Figure 8. Theoretical impact models can be derived from Hertz Theory of elastic impact, and adapted to the situation of a free body impacting a half-space (Knight et al. 1977).

The following assumptions are inherent in this analysis: 1), the impactor is modelled as a free spherical body of linear-elastic material, and 2), the impacted material is assumed to be a half-space, and remains within the linear-elastic range for the duration. Based on these assumptions, and that impactor mass, size, material and velocity are predefined and invariable properties of the instrument, the principal characteristics of the impact will be affected by the surface properties as follows:

$$\text{Contact duration } T_c = k_t \left(k_i + \frac{1 - \nu^2}{E} \right)^{2/5} \quad (4)$$

$$\text{Peak force } P_{cm} = k_p \left(k_i + \frac{1 - \nu^2}{E} \right)^{-2/5} \quad (5)$$

where: E = elastic modulus of the half-space, ν = Poisson's ratio of the half-space, and the following constants can be derived from Hertz theory (Knight et al. 1977) given the assumption

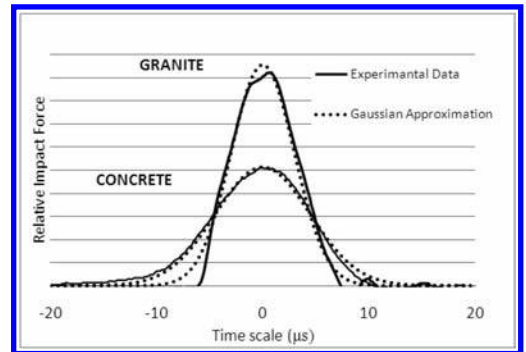


Figure 8. Comparison of the impact characteristics of two sample materials using the prototype shotcrete impact-echo device. The experimental data is superimposed with best-fit peak force (P_{cm}) and contact duration (T_c) modelled via Gaussian approximation.

of consistent impactor mass, velocity at impact and rebound resistance:

$$k_t = \frac{2.94R}{V^{0.2}} \left(\frac{5\rho\pi}{4} \right)^{2/5} \quad (6)$$

$$k_p = 1.52R^2(\rho\pi V^2)^{3/5} \quad (7)$$

$$k_i = \frac{1 - \nu_i^2}{E_i} \quad (8)$$

where: R = impactor radius
 ρ = impactor density
 V = velocity at impact
 E_i = elastic modulus of the impactor
 ν_i = Poisson's ratio of the impactor

This synopsis underpins the scientific basis for impact analysis to determine in-situ shotcrete properties, in particular the development of early age strength. As in the case of P-wave velocity, a practical implementation of this technique would preferably be complemented via selective ground-truthing, and the use of predefined, mix-specific calibrations.

6 CONCLUSIONS & FUTURE RESERCH

The paper presents some innovative concepts for the application of NDT to the quality assurance of shotcrete ground support. A review of state-of-art impact echo practice, as applied to concrete geometries, is presented. These methods show potential for application on shotcrete tunnel lining with hardware adaptations for rough surfaces and high operating frequencies.

The ultimate objective of the current R&D effort is to assess the viability of this technology to deliver cost-effective field measurements and data interpretation to provide estimates of layer thickness, bond quality and early age strength. The technology underlying these separate procedures are at varying stages of maturity, and require different degrees of further research and field verification. Whereas impact-echo thickness testing has been applied in practice over the past 25+ years, other techniques such as bonding evaluation is, in its current form, an emerging technology.

The author is currently undertaking field trials at mine sites throughout Western Australia, in order to evaluate and optimize the hardware configuration for the challenges associated with testing on uneven surfaces, and with hard rock substrate.

REFERENCES

- ACI Committee 228. 2003. In-Place Methods to Estimate Concrete Strength. *ACI Concrete Repair Manual*, American Concrete Institute, Farmington Hills, MI.
- Aggelis, D.G., Shiotani T. & Kasai K. 2008. Evaluation of grouting in tunnel lining using impact-echo. *Tunnelling and Underground Space Technology* 23 (6):629–637.
- ASTM 2004. Standard Test Method for Measuring the P-Wave Speed and the Thickness of Concrete Plates Using the Impact-Echo Method. *Standard C-1383*, ASTM International, West Conshohocken, PA.
- ASTM 2005. Standard Test Method for Flexural Toughness of Fiber Reinforced Concrete (Using Centrally Loaded Round Panel). *Standard C-1550*, ASTM International, West Conshohocken, PA.
- ASTM 2008. Standard Test Method for Rebound Number of Hardened Concrete. *Standard C-805*, ASTM International, West Conshohocken, PA.
- Bernard, E.S. 2008. Early-age load resistance of fibre reinforced shotcrete linings. *Tunnelling and Underground Space Technology* 23 (4):451–460.
- Gibson, A. 2005. *Advances in non-destructive testing of concrete pavements*, PhD Dissertation, University of Illinois at Urbana-Champaign, Champaign, IL.
- Gibson, A., & Popovics, J.S. 2005. Lamb Wave Basis for Impact-Echo Method Analysis. *Journal of Engineering Mechanics* 131 (4):438–443.
- Knight, C.G., Swain, M.V. & Chaudhri, M.M. 1977. Impact of small steel spheres on glass surfaces. *Journal of Materials Science* 12 (8):1573–1586.
- Lee, H.K. 2003. Velocity-Strength Relationship of Concrete by Impact-Echo Method. *ACI Materials Journal* 100 (1):49.
- Malhorta, V.M. & Carino N.J. 2004. *Handbook on Non-destructive Testing of Concrete*. Second Edition ed. Boca Raton, FL: CRC Press.
- Pessiki, S.P. & Johnson, M.R. 1996. Nondestructive Evaluation of Early-Age Concrete Strength in Plate Structures by the Impact-Echo Method. *ACI Materials Journal*, Vol. 93, S. 260–271.
- Popovics, J.S., Cetrangolo G.P. & Jackson N.D. 2006. Experimental Investigation of Impact-Echo Method for Concrete Slab Thickness. *Journal of the Korean Society for Nondestructive Testing* 26:427–439.
- Sansalone, M.J., & William B. Streett. 1997. *Impact—Echo, Nondestructive Evaluation of Concrete and Masonry*. Ithaca, NY: Bullbrier Press.
- Shokouhi, P., Gucunski, N. & Maher, A. 2006. Time-Frequency Techniques for the Impact Echo Data Analysis and Interpretations. In *9th European NDT Conference* Berlin, Germany.
- Song, K., & Cho, G. 2009. Bonding state evaluation of tunnel shotcrete applied onto hard rocks using the impact-echo method. *NDT & E International* 42 (6):487–500.
- Tiemann, H.D. 1909. The theory of impact and its application to testing materials. *Journal of the Franklin Institute* 168 (4):235–259.

The influence of air content on sprayed concrete quality and sprayability in a civil tunnel

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ABSTRACT: Quality mix design of fresh concrete is essential to achieve sprayability, high early strength gain, and good hardened properties. This paper presents studies of the effect of mixture design on the properties of sprayability and early-age performance. Problems with inconsistent early strength development and varying pumping pressure have forced the authors to focus on fresh concrete properties, on flowability characteristics, and on the influence of air entraining admixtures to improve performance. Five different levels of air content were tested in an attempt to achieve better sprayability, rapid early strength gain, and better quality sprayed concrete. Both fresh and hardened concrete properties were studied. The levels of air entraining admixture dosage varied from 0.3 to 4.2 kg/m³. The tests showed that requirements with respect to air content are fulfilled with dosages of 0.5 to 1 kg/m³ air entrainer, usually resulting in an air content after spraying of about 5%. At a later date, tests on air content distribution and the Borås freezing test will be done.

1 INTRODUCTION

The construction company Veidekke Entreprenør AS is constructing a railway tunnel at Lysaker, near Oslo, Norway, where rock stabilisation is undertaken with macro-synthetic fibre reinforced sprayed concrete. Problems with inconsistent early strength development and varying pumping pressure forced us to examine fresh concrete properties, focusing on flowability characteristics and on the influence of air entraining admixtures. The concrete plant at Sjursjøya, Unicon was highly interested in the trials and cooperation worked very well.

Problems with inconsistent strength development resulted in areas of sprayed concrete which displayed very high early strength and other areas that displayed very low early strength. Penetration measurements showed that different areas of the wet-mix surface did not set at the same rate. Early strength measurements (NCA, 2002) within a testing area varied considerably, depending on the location of the penetration needle. Over the last decade, many challenges have occurred with regard to early strength performance since the introduction of alkali-free accelerators. Early strength development depends directly on the reaction of accelerators with cement, which is influenced by interactions with superplasticisers and other admixtures. Polymer-based superplasticisers, mainly designed for use in Self-Compacting Concrete (SCC), make concretes more slump-stable by prolonging the setting time of concrete which

counteracts early strength development. Reducing the amount of plasticizer beneath 0.8% by weight of cement makes the concrete set earlier, but results in a stiffer concrete mix, which increases the pumping pressure, make it difficult to evenly distribute the accelerator in the nozzle. This uneven distribution of accelerator results in a wide spread in early strength variation. Strength measurements within a testing area may vary considerably, depending on the location for the penetration needle. So, certain areas of sprayed concrete display very high strength development while other areas display very low early strength.

In this study, the sprayed concrete mix design was changed to make the concrete more pumpable and sprayable without increasing the amount of superplasticiser. This was achieved by use of an air entrainer to give good flowability and enhance pump-ability, and additionally some “slump killing” effect after placing. The effect of micro-air stability was checked by air content measurements on fresh concrete (NS-EN 12350-7) and comparison with compressive strength (NS-EN 12390-3), to find an optimal air-content for sprayed concrete. Also, the effect of air entrainment on the frost durability will be checked in later tests.

One is forced to ask: “Does an air-entraining agent have any function in wet-sprayed concrete? Will air entrained sprayed concrete be difficult to pump? Is sprayed concrete placed with such a force that the air-entraining effect is negated?” These are common questions that need to be addressed.

The American Concrete Institute Committee on Shotcrete recommends air entraining in wet-mixed sprayed concrete: “Wet-mix concrete that will be exposed to freeze-thaw cycles should be air-entrained. Air entrainment tends to make some mixes more workable and may reduce rebound.” On the other hand, it is pointed out that “a significant quantity of the entrained air in wet mixtures is lost in spraying” (ACI Committee 506).

It appears to be commonly accepted that air content is reduced upon spraying in the range of 50 to 70%. According to field experience and studies done by Morgan (1989) large amounts of air are lost due to compaction. Typical air contents between 12% and 20% fall to between 3% and 6% after compaction during spraying. Lower fluidity and reduced slump are instantaneously obtained by the compaction effect of sprayed concrete, air loss upon impact is therefore called the “slump-killing effect”.

Chan et al (2002) stated that an as-batched air content of approximately 8 to 10% is used to achieve an as-sprayed air content of 3 to 5%, a reduction in air content upon impact of 50–70%. Jolin & Beaupre (2003) and Morgan et al (2004) stated that a reduction in air content will occur from 7–10% in fresh mix to 4% after spraying, a reduction of 50–60%. Among others, these authors stress the importance of a definite advantage in using a high air content during batching of concrete, resulting in a reduction in slump in the in-place sprayed concrete. This *slump-killing* effect helps reduce the amount of accelerator required to provide slump reduction and shotcrete adhesion.

2 PROJECT OVERVIEW AND ON-SITE TESTS

The Lysakertunnel is a railway tunnel between Lysaker and Sandvika in the municipality of Aker-shus, south of Oslo, Norway. The excavation is by drill and blast. Ground support is based on rock bolts and fibre reinforced shotcrete. Early strength development is essential for effective ground support. The shotcreting was executed with a Meyco Roadrunner robot. The shotcrete wet-mix was conveyed pulsation-free by twin-piston pumps in a dense stream through hoses to the nozzle. Alkali-free accelerator was added to the sprayed concrete at the nozzle. All tests were performed with the same robot and the same skilled operator. A shotcrete mix B35M45, with a characteristic compressive strength of 45 MPa (based on cubes) and a water/cement ratio of 0.45 (including accelerator), was specified throughout the tunnel and was used in all the tests. The concrete composition is shown in Table 1. The cement

Table 1. Concrete composition (specified and actual values).

Concrete materials (kg/m ³)	
Norcem Std FA Cement	480–496
Micro silica	20–21
Water	168–176
Sand	1464–1492
Superplasticiser Glenium 151	3.5 (0.72%)–3.9 (0.80%)
Air entrainer Micro Air 100	0–4.3
W/C ratio	0.42

used for all shotcrete mixes was a CEM II/A-V 42.5R type, Norcem Standard FA (NS-EN197-1) Cement, and the micro-silica came from Elkem Materials. A polymer superplastizising agent BASF Glenium 151, and air entrainer Micro Air 100 (1:19) from BASF was used. The alkali-free accelerator used was BASF SA 162/170. The aggregate used was non-alkali reactive (0–8 mm). The fibres were a macro-synthetic fibre (Barchip Kyodo 48 mm).

2.1 Fresh concrete mix design

The water/cement ratio is the most important parameter controlling sprayed concrete quality and performance under long term conditions. The water/cement ratio is limited to a maximum value 0.45 for road and railway tunnels. Superplasticiser was added to obtain a low water/cement ratio and good pumpability. Slump values between 170 and 240 mm were tested. To minimize the retarding effect of superplasticiser, the total amount of superplasticiser in the mix was kept below 0.8% by weight of cement. Additionally, an air entraining admixture was added to enhance the workability (slump) and sprayability of the sprayed concrete. The composition of the fresh concrete mix is shown in Table 1.

2.2 Workability of fresh concrete

Workability is important to the quality of the sprayed concrete. Too high a slump often causes high rebound and sagging of the freshly applied sprayed concrete. Mixtures with low slump give high pumping pressures and slugging, and it is difficult to obtain an even distribution of the accelerator. Air entrainment enhances the workability (slump) of the shotcrete. Slump was tested according to NS-EN 12350-2.

2.3 Air content

Air content in fresh concrete is commonly measured in the field using the pressure method

(NS-EN 12350-7). This method is based on the principle that the only significantly compressible component of fresh concrete is the air. Pressure is applied to the sample to compress the entrained air in the pores. The pressure-meter uses the change in known volume of air to determine the air content of the mix. The air content of fibre reinforced concrete is measured at the concrete plant right after batching, at the tunnel site before spraying, and at the tunnel wall right after spraying (sample from the tunnel wall). The air content after spraying was measured by digging out the in-place sprayed concrete from the tunnel wall, and compacting it in the air pressure meter, as for fresh concrete. For earlier tests, two test-methods were evaluated: a comparison study of both, spraying directly into the air pressure meter (nozzle 90 perpendicular to the air pressure meter bottom) and digging out the in-place sprayed concrete from the tunnel wall. Both methods showed comparable results. All air content measurements from the tunnel site represented a mean value of two values each.

3 RESULTS AND DISCUSSION

3.1 Compressive strength

Cube specimens for compressive strength measurements were taken at the delivery point immediately before spraying (NS-EN 14488-1). Additionally, cores were drilled from the sprayed lining in the tunnel. Core specimens for compressive strength measurements were tested according to NS-EN 12390-3. Cylinder strength was corrected for height/diameter relation, according to this standard. Each of the compressive strength results (cubes and drilled cores) exceeded the specified B35 requirement, see Figures 1 and 3.

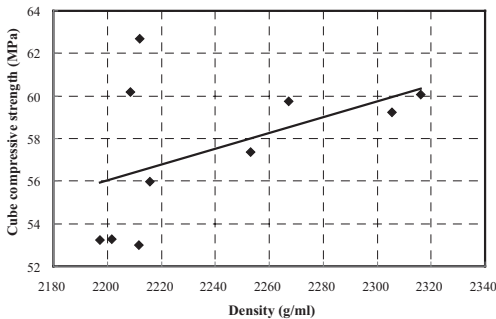


Figure 1. Cube compressive strength and density results.

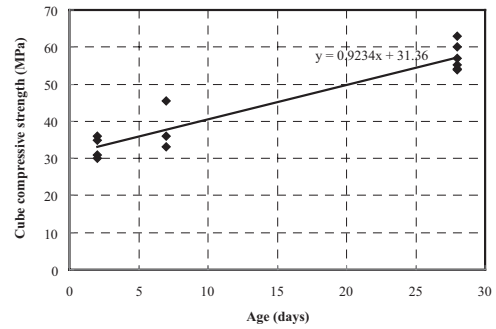


Figure 2. Cube compressive strength development.

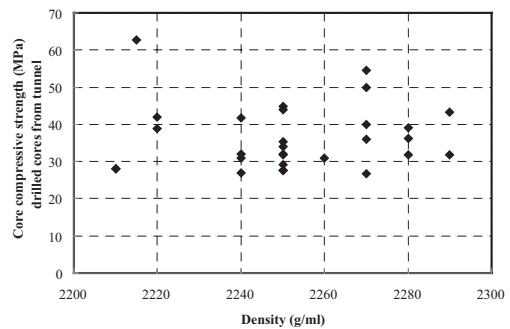


Figure 3. Core compressive strength and density results.

Concretes with high air contents seemed to lose some air during preparation of the cube samples, this may explain the higher strength/higher densities than expected due to measured air contents.

The Compressive strength development of cubes was, as expected, 50% strength achievement after 2 days (Figure 2). Final compressive strength of the cubes was between 53 and 63 MPa. Cores drilled from the sprayed lining in the tunnel reached a lower strength compared to the specimens stored in laboratory conditions, but still reached minimum 28 MPa, which is equivalent to 35 MPa with a correction factor for drilling. The strength/density relationship for drilled cores from the tunnel lining is shown in Figure 3. It appears that strength is not influenced by density. This may be due to the fact that the macropores after spraying are lost due to compaction, and the micro pores appear not to reduce the strength in the same manner.

3.2 Air content

Air content measurements were carried out after batching, after transport to the tunnel site, and after spraying. Figure 4 shows how air content changed during transport. The change in air content due to spraying compaction is shown in Figures 5 and 6.

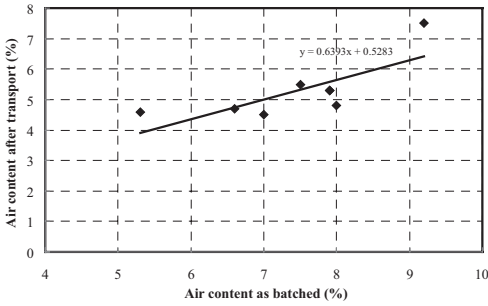


Figure 4. Influence of concrete transport on air content.

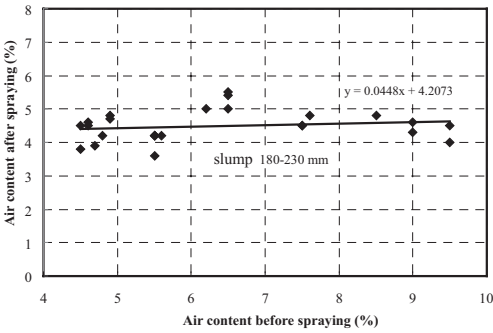


Figure 5. Influence of spraying compaction on sprayed concrete air content.

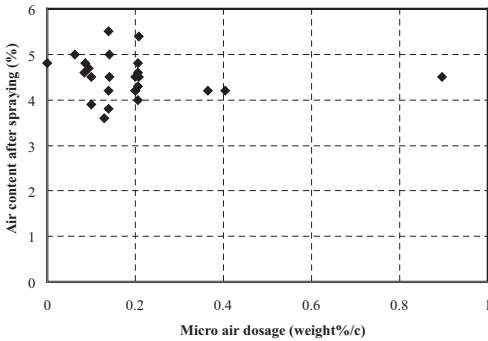


Figure 6. The relation between air-entrainer dosage rate and the amount of air after spraying.

All specimens ended up with an air content between 4% and 5% after spraying that appeared to be independent of the air entrainer dosage rate and air content before spraying. The air loss due to compaction varied between 0% and 70%, see Figure 7. The air loss was 0% to 20% for concrete with 4% to 5% air content before spraying. Concrete with high air content, above 5% up to 9% may loose up to 70% due to compaction.

Further studies will be done on relation of air content/compressive strength—both for cubes and cores.

3.3 Early strength, slump and pumping pressure

Slump measurements on the fibre reinforced sprayed concrete were done after batching and after transport to the tunnel site, right before spraying. There appeared to be no direct relation between slump before spraying and air loss due to compaction, see Figure 8.

Early strength measurements were executed on the tunnel wall one hour after spraying (NCA Publ. No. 7). A direct relationship appears to exist between early strength achievement and how well accelerator was mixed into the concrete. High pumping pressure results in low early strength development which we have assumed is due to inhomogenous mixing of accelerator, as seen in Figure 9.

A stiffer concrete mix, which increases the pumping pressure, makes it difficult to evenly distribute

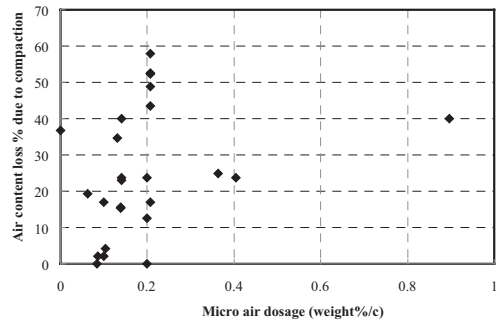


Figure 7. The relation between air-entrainer dosage and air content loss percentage due to compaction.

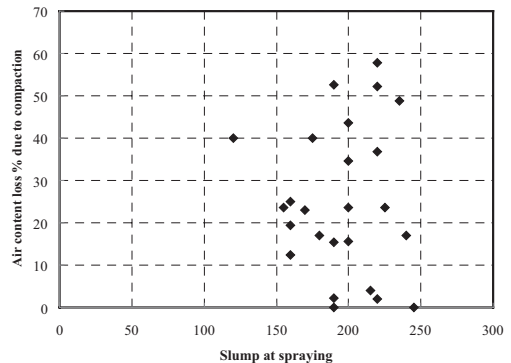


Figure 8. The relation between concrete slump before spraying and air content loss percentage due to compaction.

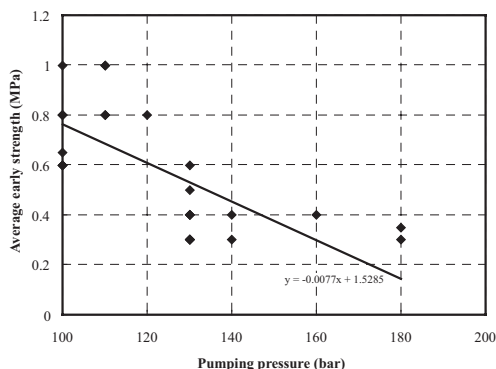


Figure 9. Early strength as a function of pumping pressure influenced accelerator homogenization.

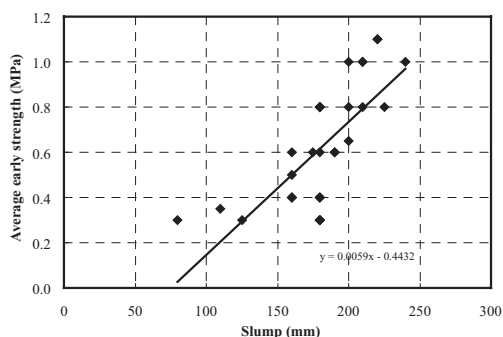


Figure 10. Early strength as a function of concrete workability, slump value.

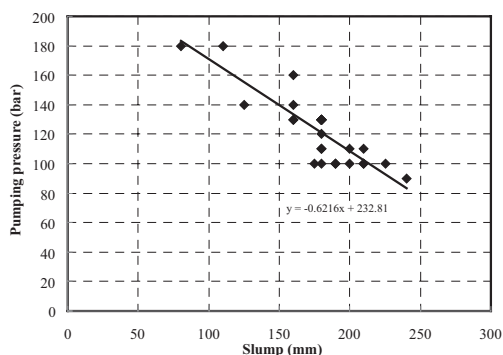


Figure 11. Pumping pressure as a function of concrete slump.

the accelerator in the nozzle. This uneven distribution of accelerator results in a wide spread in early strength values, see Figure 10.

Strength measurements within a testing area varied considerably. So, certain areas of sprayed concrete displayed very high early strength while

other areas very low early strength, resulting in a low mean strength value. Figure 11 sums up the relationship between concrete workability and pumping pressure.

4 CONCLUSION AND FURTHER RESEARCH

The Lysaker tunnel project was chosen for a full-scale laboratory and on-site test for reinforced wet-shotcrete. To solve problems with inconsistent strength development the sprayed concrete mix design was changed to make the concrete more pumpable and sprayable without increasing the amount of superplasticiser. This was achieved using an air entrainer to give good flowability and additionally some slump killing effect during placing.

Air entrainment, even at low dosages, led to excellent pumping characteristics. The concrete developed a creamy consistence and the need for superplasticiser was reduced. All the specimens ended up with an air content between 4% and 5% after spraying independent of the air-entrainer dosage rate and air content before spraying, well within freeze-thaw requirements.

Sprayed concretes with low air contents before spraying of about 4–5% displayed an air loss due to compaction of only between 0 and 20% after spraying, resulting in a final air content of 4–5%. An entrained air content of 4% will always remain in the sprayed concrete independent of the spraying compaction. These concretes with low air entrainments exhibited the required excellent pumpability and sprayability.

Sprayed concretes with high air contents before spraying lost about 60–70% of the air due to spraying compaction, resulting in air contents of 4–5% after compaction, which is comparable to results shown by Morgan et al (2004) and others. The slump-killing effect obviously resulted in a reduced need for accelerator within the sprayed concrete to stick to the tunnel wall, but the accelerator reduction automatically results in lower early strength achievement. Improved pumpability due to air entrainment results in a homogeneous and well distributed accelerator distribution within the sprayed concrete. This results in uniformly higher average early strength achievement.

REFERENCES

- ACI Committee 506. "Guide to Shotcrete", American Concrete Institute, Farmington Hills, Mich.
- Chan, C., Heere, R. & Morgan, D.R. 2002. "Shotcrete for Ground Support: Current practices in Western Canada", *Shotcrete Magazine*, Spring.

- Jolin, M. & Beaupré, D. 2003. "Understanding Wet-Mix Shotcrete: Mix Design, Specifications, and Placement", *Shotcrete Magazine*, Summer 2003.
- Morgan, D.R. 1989. "Freeze-Thaw Durability of Shotcrete", *Concrete International*, Vol. 11, No. 8, August, pp. 86–93.
- Morgan, D.R., Ezzet, M. & Pfhof, C. 2004. "Rehabilitation of the Seawall at Stanley Park Vancouver, BC, Canada with Synthetic Fibre Reinforced Shotcrete", *Shotcrete: More Engineering Developments*, Proceedings of the second International Conference on Engineering Developments in Shotcrete, Cairns, Queensland, Australia.
- Norwegian Concrete Association Publication No. 7, 2002. Sprayed concrete for rock support—Technical Specification, Guidelines and Test Methods (NCA Publ. No. 7).
- NS-EN 197-1 Cement. Composition, specifications and conformity criteria for common cements.
- NS-EN 12350-2 Testing fresh concrete—Part 2: Slump test, 2000.
- NS-EN 12390-3 Testing hardened concrete—Part 3: Compressive strength test of test specimens, 2001.
- NS-EN 12390-3 Testing hardened concrete—Part 7: Density of hardened concrete, 2001.
- NS-EN 12350-7 Testing fresh concrete—Part 7: Air Content—pressure methods, 2000.
- NS-EN 14488-1 Testing sprayed concrete, Part 1: Sampling fresh and hardened concrete, 2005.

Shotcrete as a good looking final finish—it *is* possible!

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ABSTRACT: Shotcrete has long been used in civil works for free-form structures such as swimming pools and vertical construction of embankment walls and slope stabilisation. The majority of this type of work is still applied by hand. One of the chief limitations to shotcrete being used as a final finish is the aesthetic quality of the finished product. To overcome this, a variety of surface finishes have been developed by leaders in the industry including trowelled, coloured and sculptured finishes. However, due to some well documented examples of poor shotcrete construction many urban planners, government departments, builders, consultants and perhaps also the general public, consider shotcrete to be an unattractive final finish. This paper examines the recent history of the use of shotcrete in public infrastructure in Australia and examines how the correct application of skill and technique can avoid these problems.

1 INTRODUCTION

Shotcrete was first introduced in 1907 in America as hand applied dry spray, commonly known as *gunite*. Wet-mix shotcrete was later introduced in 1955 which increased pumping volume capacity and thus production rates. As such, wet-mix was and remains perhaps more difficult to control and finish than gunite. In the late 1960s robotic equipment was introduced and is prevalent today for overhead spraying in mines and tunnels. Around 800,000 m³ of shotcrete is sprayed in Australia per year.

Given its ease of application, suitability for free form construction and elimination of any formwork other than a surface upon which to spray, shotcrete became and remains perfect for construction of swimming pools, tanks, water features, shoring, underpinning, dam spillways, and ground stabilization including overhead application as well as many other structures.

2 LESSONS IN HOW NOT TO SPRAY SHOTCRETE

2.1 M2 Motorway, Sydney NSW

A construction contract was awarded to a large civil contractor. Work commenced on this 21 km long NSW project in 1994/95 and was completed in 1997.

Many, often deep, vertical and steep-battered excavations were required to allow the road and tunnel associated with this project to pass through this challenging site. The retaining systems employed were generally soil nailed or

anchored walls constructed in a top-down excavation sequence. Once the soil nails and anchors were installed, the areas were meshed and shotcrete was sprayed with a natural shotcrete (grey) gun finish while in some areas coloured oxide was added. The excavation staging was poor resulting in highly irregular horizontal joints between shotcrete sprays. In addition, the finish was “off-the-gun” which is very basic and cheap and the general quality of the finished product was poor.

As a result it is possible to see the ribs of the mesh in places (Figure 1) and permanent anchor heads were not recessed into the shale/rock. Large knobs of shotcrete were sprayed over the top of each resulting in hundreds of unsightly lumps



Figure 1. M2 anchored walls—poor staging, poor spraying, extensive cracking.

sticking out of 12 m high walls about 300 m along both sides of the motorway (Figures 2 and 3). Over the last 12 years significant cracking has appeared and in some areas the shotcrete has come away from the base as no keying in was provided for (Figure 3). It is fair to say that the shotcrete construction on the M2 Motorway is an eye-sore. Worse still, thousands of motorists pass the work every day, leaving the poor standard of work etched into the public psyche.

It is not clear whether the specification, the superintendent or contractor employed was at fault on the M2 project, but it seems likely that all these elements were in some way responsible. It is believed that as a result of this project the Roads

and Traffic Authority of NSW instructed that shotcrete not be used as a final finish or, wherever possible, be limited in use. RTA Shotcrete Design Guidelines (RTA, 2005) certainly suggest that this is true although it does not specifically state that the M2 is to blame. Whilst the document does not suggest shotcrete must be banned from all future roadways it says that it should be avoided wherever possible. The document states:

“The best strategy in dealing with shotcrete in terms of cost, safety, appearance and environment is to adopt the following hierarchy:

1. Avoidance
2. Minimization
3. Improve Appearance”

In retrospect, if point No 3 was more fully considered back in 1994 then perhaps this document would not exist! There are, of course, other poor examples of shotcrete (see F6 freeway, Sydney, Figure 4).

Generally the most damaging instances of poor finish for the shotcrete industry are those examples on the sides of roadways as they are open to public scrutiny. These works are characterized by:

- Generally large spans of gun finish (the most basic version of shotcrete)
- Poor excavation staging
- Poor application techniques
- Poor shotcrete quality resulting in excessive cracking.

Despite the public relations disaster of the M2 motorway project, the use of shotcrete has continued. One increasingly popular method is the use of vertical soil nail walls for embankment stabilisation. However, the shotcrete employed in the construction is often hidden from public view.

The construction method now commonly employed is as follows: a top capping beam is



Figure 2. M2 motorway anchored wall—mesh ribbing is evident suggesting inadequate cover to reinforcement.



Figure 3. M2 motorway—another poorly staged and sprayed wall with excessive cracking.



Figure 4. F6 freeway, Sydney—poor staging, poor spraying, overspray and excessive cracking.

installed, the soil nail walls are constructed in stages and sprayed to a gun finish, a bottom beam is built, and then concrete pre-cast panels are installed over the shotcrete. This requires large cranes, massive trucks, and boom lifts to complete. This is a very expensive process to eliminate unsightly shotcrete. Yet it is totally unnecessary if the shotcrete is well constructed and finished. The result of the current approach using pre-cast elements is: a tripling of the cost, a doubling the time to construct, and a site congested with trucks, cranes, boom lifts, additional labour and all the attendant safety issues.

3 WHAT SHOULD HAVE BEEN DONE AND WHAT CAN BE DONE

The shotcrete should have been specified with a quality final finish. With the correct application of skill and technique, almost any quality of finish is achievable. Pre-construction trials should have been carried out to prove the final finish was acceptable. Construction time would have been halved. The improved efficiency would have been reflected in much lower project costs. The reputation of shotcrete would also have been restored.

It is worth noting that, at the time of writing, a A\$550 m upgrade of the M2 is due to commence in 2010 (Earthmoving & Civil Contractor, 2009).

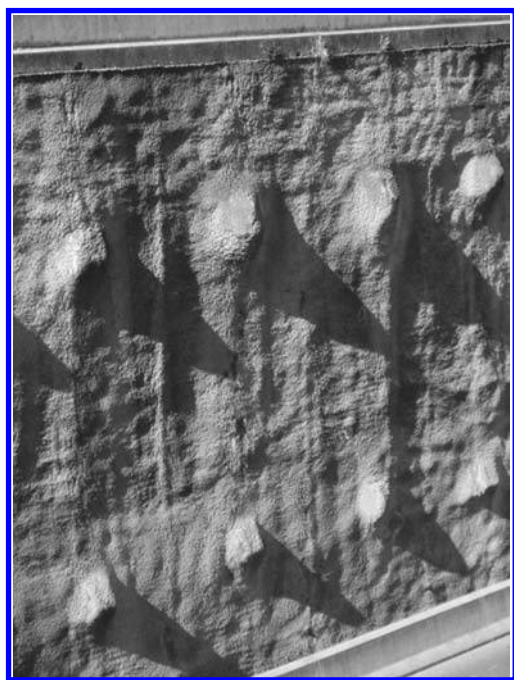


Figure 5. These walls should be re-sprayed.

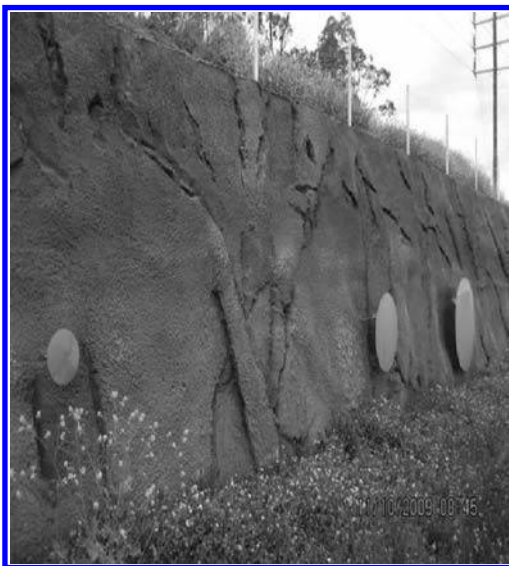


Figure 6. An example of a sculptured shotcrete wall, Geelong Ring Rd, Victoria.



Figure 7. A simple well-sprayed gun finish, Mount Conjola NSW.

The plan is to widen the roads and tunnels adding an extra lane in either direction (where possible) and construction of various on and off ramps. The design and method of construction is not yet clear but it seems likely that some of the existing shotcrete batters and walls may be re-cut to allow extra space.

Shotcrete soil nail walls remain an excellent solution for these cuts and thus would provide a great opportunity for the above advice to be considered and hopefully adopted. Of course this upgrade also offers the owner the opportunity to fix the remaining offending areas of poorly finished shotcrete cost-effectively as construction works proceed

(Figures 5, 6 & 7). Many simple shotcrete options could be adopted and a large variety of aesthetically pleasing finishes could be specified.

4 THE WAY FORWARD

4.1 Some recent shotcrete projects

Shotcrete has been adopted as a final finish on a number of new rail corridors in Australia. Road and tunnel builders are once again allowing skilled contractors the opportunity to construct various final finished shotcrete walls (Figures 8 to 10).

At Shannon Creek Dam in Queensland, the builder and the shotcrete contractor worked with the Department of Commerce to replace the designed formed and poured walls with shotcrete construction. Pre-construction testing included



Figure 8. Finished walls with joints, LBBJV—Clem 7 for cycle ways (formerly North South Bypass Tunnel, Brisbane).



Figure 9. Finished walls with joints, LBBJV Clem 7 under bridges.



Figure 10. Simulated rock finish above pre-cast Clem 7 tunnel project, Brisbane.



Figure 11. Shannon Creek Dam spillway and stilling basin.

tests on compaction, strength, and finish. The method gained approval and 3200 m² of shotcrete walls were constructed. These were up to 300 mm thick with two layers of reinforcement and up to 11 m high (see Figure 11).

5 TYPES OF FINISHES

The following types of surface finish are available for shotcrete, listed in terms of quality of finish and therefore expense.

5.1.1 Gun finish

Also referred to as 'off-the-gun' or 'undisturbed natural finish'. Put simply the aggregate is not trowelled in, the surface is rough (but when applied

by an experienced and professional nozzleman it will be uniform) similar to a pebbledash or stipple finish (see Figure 12). This is the cheapest finish as it requires minimal labour.

5.1.2 Screed finish

Guide lines are set up to the required height and face and a screed is used to cut the wall back to the required line. Aggregate drag marks occur but when an experienced screeder with the correct tool attacks the wall it can be closed up and finished fairly well.

5.1.3 Wood float and sponge finish

After screeding, wood floats are used to close the face and once the shotcrete has set (but is still green) a sponge is finally used to produce an off-render finish (Figures 13 to 15).



Figure 12. Off-the-gun or natural finish—Shannon Creek Dam, Grafton NSW—Embankment stabilisation.



Figure 13. Wood float and sponge finish with dummy joints at 3 m intervals—cycle way, Clem 7 Brisbane.



Figure 14. Wood float and sponge finish over Gun Finish along side pre-cast panels Cycle Way Clem 7 Tunnel Project Brisbane.



Figure 15. Wall completed with block work, WFS shotcrete and pre-cast panels, cycle way, Clem 7 tunnel project, Brisbane.

5.1.4 Steel trowel

Once screeded the face is steel trowelled leading to a glassy smooth finish (see Figure 16). It is often more difficult to achieve a superior finish with this method unless the shotcrete and finishers are of the highest quality. It is generally a less forgiving final finish than wood, float and sponge as all elements—shotcrete, crew, prep work and weather—must be perfect to allow an excellent finish.

5.2 Joints

In most designs crack inducement is generally not specified where a gun finish is used. Dummy joints should be encouraged simply because like any concrete structure it will crack somewhere. It is recommended to install a dummy joint at no more than 3 m centres in order to induce cracking (Figure 17). Where panels of shotcrete are higher than 3 m then horizontal joints should also be considered.



Figure 16. Steel trowel finish for an in-ground trough.

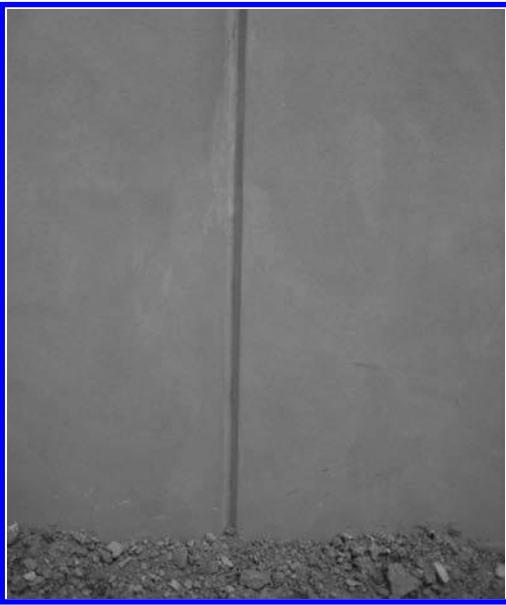


Figure 17. Dummy joint in shotcreted wall.

5.3 Oxides and pigments

Coloured oxides and pigments can be added in an attempt to match the finished colour with surrounding areas. This can result in a very aesthetically pleasing finish, especially when surrounded by rock, sandstone or soil. The drawbacks are that they must be uniform throughout and, as sands and aggregates in shotcrete can vary from day to day or even from load to load, colours cannot be guaranteed. Many concrete suppliers require client's written acceptance that this may occur before they will agree to supply.

5.4 Simulated rock or block finish

Shotcrete, generally without coarse aggregate, can also be cut and molded then coloured to offer rock, sandstone, shale or block-work finishes. These finishes, when completed by competent artisans and matched to surrounding finishes, cannot generally be identified as shotcrete. In my opinion, especially when used with landscaping, they really do offer an aesthetically pleasing retaining solution (Figures 18 and 19).

When specifying walls for appearance a very important point to note about large spans of concrete and finished shotcrete is that they offer a very large blank canvas for graffiti artists. If you look at any rock face, be it simulated or natural, you will find that they are ignored by our 'alternative artists'! Gun finishes are also generally left alone. This definitely presents an advantage over the flat or smooth finishes typical of poured concrete.

The Figure 20 shows the recently completed Lane Cover Tunnel project in Sydney. This was a very challenging site in terms of the civil works.



Figure 18. Simulated rock finish at Kirribilli, Sydney. No graffiti problem after 7 years!



Figure 19. Simulated rock using shotcrete on side of M7 Freeway Sydney NSW.



Figure 20. Shotcrete covered with pre-cast panels partly covered with creepers, Lane Cover Tunnel project, Sydney.

New east- and west-bound lanes were built within an extremely tight corridor. Many very high and vertical excavations were required. The solution adopted throughout was top down excavation and staged shoring using soil nail and shotcrete walls. Basic gun finish was specified with all of it covered by pre-cast concrete panels.

After the pre-cast panels were installed garden beds and Jersey barriers were also added. Plants have now grown up the walls which soften the pre-cast concrete panels. It was, of course, possible to simply grow the vegetation over the shotcrete saving time and cost in the secondary construction and installation of the panels.

6 OTHER EXAMPLES OF SOFTENING OF LARGE SPANS OF SHOTCRETE

The famous Victoria Peak in Hong Kong allows the visitor the opportunity to walk around the top of this scenic area and enjoy the breathtaking view. In order to achieve this a large amount of stabilization was necessary in the form of dry sprayed shotcrete walls with many outlets for weep drains allowing hydrostatic pressure relief from the apparent ground water issues. The gun finished shotcrete is therefore pitted with holes but vegetation has been encouraged throughout. This can be achieved by spraying an organic material (such as yoghurt) over the gun finish to encourage moss and vegetation to flourish. The gun finish is thus camouflaged in part by the growth and the shotcrete is unnoticed by the passerby.



Figure 21. Victoria Peak, Hong Kong.



Figure 22. Creepers covering shotcrete wall in Singapore.

Another example (Figure 22) is in downtown Singapore where bored piles with gun finished shotcrete in-fills are simply covered over time with creepers and vines again offering the same softening to the large span of shotcrete. This is not dissimilar to the strategy seen on the Lane Cover Tunnel walls above.

7 SPECIFICATION FOR A QUALITY SHOTCRETE PROJECT

The following points must be considered when developing a specification for a high value project demanding quality shotcrete. Firstly, consider the available budget and research the finish options. Specify walls for appearance but remember they are large spans/canvases and consider what they may look like in the future. Specify that the contractor must have the required experience and request evidence that all personnel, especially the nozzle men and finishers, are experienced before

they undertake these works. Request proof that the contractor has sufficient labour and high quality equipment to ensure project programme requirements are met. Specify that the shotcrete mix supplier must not only provide the required mix to meet specified design but also that it be of high quality at all times to allow ease of pumping, spraying and finishing.

Specify that the contractor must pass a rigorous pre-construction trial including: setting up a test panel reflecting what is to be built including specified reinforcement, hand spraying and finishing the specified mix with the intended qualified nozzle man and finishers. The shotcrete supplier's technician should attend the trial observing and noting any mix issues in respect to pumping (blockages, loss of water in the mix, increased pump pressures) and spraying (excessive rebound, sagging on the reinforcement, fall outs). The technician must also note finishing characteristics with regard to retardation or acceleration (long or short setting times which both hinder finishing and therefore the final appearance).

Specify realistic testing of the mix. As a minimum, cylinders must be taken from trucks for compressive strength. Cores can also be taken from sprayed panels to confirm compaction, density and in-situ compressive strength. Further testing required under the specification might include durability or toughness for fibre reinforced shotcrete.

Specify that all overspray is to be removed and limited wherever possible. Specify cover to reinforcement and that all surfaces should be wetted down at all times. Specify the method of curing. Curing compounds are generally recommended for ease of application. If the surface is to be painted a water based compound should be used. Specify that written approval of the finish by client is required before commencement of the works. In the absence of an Australian standard for shotcreting, reference should be made to the following for assistance when specifying shotcrete design, finish, application, testing etc: "Shotcreting in Australia" (2008) and RTA B82 Specification—*Shotcrete Work* (2006).

8 CONSTRUCTION PLAN FOR A SHOTCRETE PROJECT

Ensure that you price the works with sufficient allowance for the specified pre-construction works. Allow enough time and money to assure a high quality finish can be achieved. Undertake trials for shotcrete quality and finish to be approved by the client. Plan and set the works up well. The following target profile (Figure 23) suggests ideal heights, staging and benching. This allows productive and quality shotcrete placement and finishing.

Excavate ground to a given level horizontal line (this will ensure you have a straight joint in the shotcrete) and a maximum 2.5 m drop per level. This way a standard sheet of mesh can be used and eliminates the need for scaffolding or access equipment which is cumbersome and reduces productivity. Install reinforcement, drains, soil nails and all screed lines correctly. Reinforcement must be pinned and tied back rigidly and to the correct position for cover. Poorly fixed reinforcement with excessive cover to screed lines increases the risk of fallouts and results in lost time, poor finish and possibly failure of the structure during spraying.

Timbers or steel sections should be installed at required position of construction joints (generally 300 mm above the bottom of the mesh to offer lapping of mesh to the proceeding level) and in line with screed lines above. Spray good quality shotcrete with highly trained craftsmen who work as a team, take their time and give attention to detail. Reject poor quality shotcrete. Immediately contact the supplier and communicate any issue to minimize disruption. It is poor practice to continue with a poor quality shotcrete mix. This will cost time and money and will affect the final quality of finish.

Install joints as specified. Finished walls should have dummy joints as a minimum requirement. This helps to break up long stretches of shotcrete

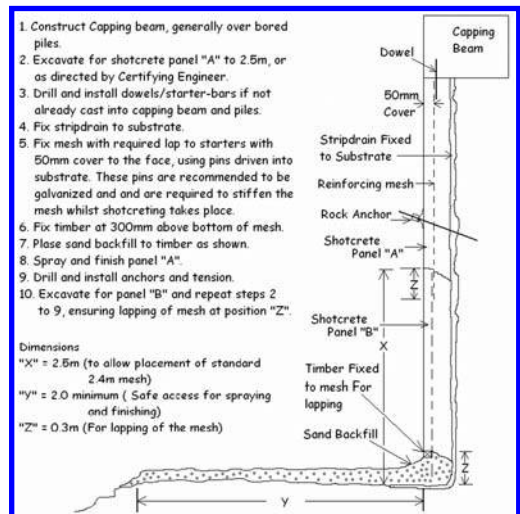


Figure 23. Suggested target shotcrete profile sequence for top-down construction. [Disclaimer: The intention of this profile is to demonstrate the best case scenario in terms of safety, productivity, cost and final finish when constructing walls in a top down nature. Final design and staging should always be checked and directed by a qualified engineer prior to commencement.]

and also helps to induce cracking. Finish the walls to a high standard with the crew working as a team at all times. Ensure that all overspray is cleaned off surrounding surfaces. Where possible, screen surrounding areas prior to commencement to prevent over-spray that requires subsequent cleaning. Avoid spraying in very hot temperatures and, if unavoidable, start and finish early. Extremely cold temperatures can also cause problems. Ensure the substrate is wetted down thoroughly and continually throughout the day as the moisture will be drawn out rapidly from dry substrates. When you sponge a wall, ensure your sponge is new, constantly washed out in clean water and is never too wet. Run a damp sponge over any wet sponge marks and repeat until the required final finish is complete.

Have the supervisor check all the work and fix any areas that are not up to standard. Apply a curing compound that does not stain the shotcrete and can be painted over if required. Ensure the works area is fully prepared and cleaned of all the previous days' rebound and screed waste. Screen any necessary areas from overspray. No matter how well your walls are finished, over-spray will ruin the appearance and spoil all the hard work. Clean sites are safe and most productive!

9 CONCLUSIONS

Where poor examples of finished shotcrete work are evident, it is generally the case that the constructor has:

- Not researched the available options for finishing,
- Specified it poorly,
- Calculated the budgets and programmes poorly,
- Produced the shotcrete poorly,
- Applied it poorly.

Shotcrete is more than capable of taking its place with most of the other structural retaining options in terms of finish as long as it is correctly specified and supervised. Given the same respect and attention to detail as competing options, shotcrete is at least as good as a final finish.

A greater understanding, awareness and recognition will be achieved with the completion of more examples of projects incorporating quality shotcrete finishes. Those that have continued to consider shotcrete in these instances should be applauded whilst the culprits will hopefully be purged from the industry. In the absence of any Australian standards for shotcreting the Australian Shotcrete Society's "Shotcreting in Australia" (2008) is a vital tool both locally and internationally and essential to any interested party.

REFERENCES

- Earthmoving & Civil Contractor Weekly Newswire, Oct 15 2009, "M2 Upgrade".
- Roads and Traffic Authority of NSW, 2005. *Shotcrete Design Guidelines*, RTA/pub.05.116 current in June 2005.
- Roads and Traffic Authority of NSW, 2006. QA Specification B82—*Shotcrete Work*—Edition 2/Rev 0 RNIC-QA-B2.
- Shotcreting in Australia*, 2008. Concrete Institute of Australia and Australian Shotcrete Society, Sydney.

Shotcrete research and practice in Sweden—development over 35 years

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ABSTRACT: In 1973 a large research project on shotcrete in hard rock tunneling was started by the Royal Swedish Fortifications Administration (FortF) and the Swedish Rock Mechanics Research Foundation (BeFo). That project was also the starting point for me on a long journey through a totally new landscape—the underground. In this paper the development of shotcrete practice in Sweden and its connection to research will be described. In 1973 neither bond testing, rock anchored reinforced shotcrete, fibre reinforced shotcrete or modern accelerators were known. Nor were high speed trains creating considerable loads from under- and overpressure on tunnel linings. The mode of action of shotcrete drains was not explored either. Today there is an increased need for better methods of crack prevention and improved knowledge regarding shotcrete subjected to blasting vibrations and the time-dependent development of material properties in shotcrete.

1 AUTHOR'S STATEMENT

1.1 General

I have been involved in shotcrete research since 1973. This has led me to believe that a general review paper is of interest to the shotcrete community. It is based on my experiences connected with shotcrete in hard rock, but it is not the complete history of shotcrete in Sweden.

1.2 References

In this paper some facts cannot be supported by references since they are based on my own experiences or on personal information. I also believe that too many references in this type of text would make it difficult to read. Important scientific reports and papers have, however, been referenced.

2 THE SIXTIES

2.1 Design

In the nineteen sixties shotcrete linings were not designed in the manner that they are today, but of course there were many ideas about the mode of action of shotcrete in hard rock. It was intended that shotcrete penetrate cracks and keep smaller or larger keystones in place thus helping the rock mass to stabilize itself. The pyramidal block, which punches through the lining, was early introduced as a model for the mode of action of shotcrete. The

shotcrete lining was normally designed on the basis of experience and the thickness of the shotcrete was normally in inverse proportion to the quality of the rock. For the worse rock conditions reinforcing mesh was introduced together with increased thickness of the shotcrete or sometimes instead of increased thickness. Rock bolts were normally not designed for interaction with the shotcrete, which seems reasonable in the cases when the shotcrete was unreinforced, but that was also the case when the shotcrete was reinforced.

2.2 Practice

All spraying was by the dry mix process. Where water ingress was observed mineral wool drains covered with plastic sheets were used. They were then covered with steel mesh and sprayed. In some cases the shotcrete was accelerated with extremely high dosages of a powder accelerator dosed with a shovel! Such shotcrete could stop the water inflow but the material soon broke down chemically. If the accelerated shotcrete was covered with shotcrete without accelerator a dangerous situation could be created in which the lining could look satisfactory from the outside although it did not bond to the rock surface.

3 THE SEVENTIES

3.1 Research

At the University of Illinois at Champaign-Urbana a comprehensive research program on

shotcrete commenced. That inspired the Swedish Rock Mechanics Research Foundation (BeFo) and the Royal Swedish Fortifications Administration (FortF) to start discussions about a Swedish research project on the interaction between shotcrete and hard rock. I was asked to be the leader of that project. After some initial discussions it was decided that a series of well controlled tests on a simple block model should be performed at full scale. The reason for this decision, among other things, was that the pyramid-shaped block model commonly in use assumed a punching shear failure of the shotcrete at the boundary of the block, but it was doubted to some extent whether this mode actually occurred in reality. The reason why full scale testing was chosen was primarily pedagogical: it was believed that practitioners would be easier to convince about the relevance of the testing if it was conducted at full scale.

A large steel stand with three granite blocks totalling 5 tonnes was constructed, Figure 1. It was possible to tilt it so that the top surface of the granite blocks could be sprayed in a vertical position. Before testing it was tilted back so that the shotcrete was on top and the middle block was pressed upwards during testing. The test rig was used over several years. Tests on unreinforced, mesh reinforced, bar reinforced and fibre reinforced plane and arch shaped shotcrete layers were performed (Holmgren, 1979, 1985a). Many of the test specimens were also rock anchored. The most important findings were that the bond between shotcrete and rock constitutes a very strong connection between the two materials and that there is a weak correlation between the thickness of the layer and its load transferring capacity over a crack. This subject is still under discussion and not everyone agrees with my finding. The tests also clearly showed that rock-anchored reinforced shotcrete has a beneficial effect both when bond is unreliable and when a plastic behaviour is required.

The findings of this project put a focus on the bond between shotcrete and rock. Hahn performed

a thorough laboratory investigation on bond between shotcrete and different rock types. His report was published in Swedish, but an English, shorter version is to be found in (Hahn & Holmgren, 1979). This study showed that very good bond (above 1 MPa) is to be expected against most of the normal rock types in Sweden except for mica schist and, of course, also excessively weak rock types.

3.2 Design

Some consultants realized the importance of bond and put requirements on bond into contracts. That caused some confusion in the beginning, but when rules had been established how to assess the test results and how the test equipment should be designed, the requirement for 0,5 MPa bond strength was accepted and it is still in force today. The value 0,5 MPa was intentionally chosen because it is a low value compared to what is practically reachable. The idea is that every contractor who does a good job shall not fail in this respect. On the other hand contractors who do not clean the rock surface or do not inform the owner about bad rock conditions should be punished by this rule. This background is important to know because today some people want to divide this value with safety coefficients of different kinds, something that is already built-in to this rather low value.

The Swedish Fortifications Administration (FortF), which had special types of caverns designed for explosive loading, decided that only "simple" linings were allowed to be sprayed without supporting design calculations. More advanced types of lining should be designed using models where the lining was supposed to carry a certain "rock load". That forced design engineers to go through all details of the linings and show by calculation that they could carry the assumed load. Thus the load carrying capacities of the rock bolts, the plates or washers on them, the reinforcement in the shotcrete between the bolts, and the shotcrete itself became balanced against each other. If the required capacity of any part of the system was found not to be inadequate then a re-design could easily be performed.

3.3 Practice

During this decade the wet mix method became more and more used. Experiments with steel fibre reinforced shotcrete were also performed. At first short fibres of cold drawn wire were used. The method of adding the fibres was also subjected to a lot of experimentation. Cold drawn fibres with wave form added with a vibrating table gave an acceptable quality of the shotcrete. One contractor

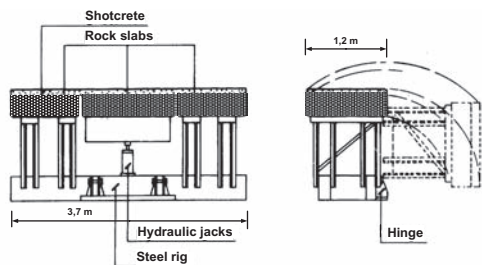


Figure 1. Large scale test rig for experimental investigation of the mode of action of shotcrete bonded to hard rock.

replaced such fibres with ones cut from a steel band in a contract with the Swedish Road Administration. That produced an extremely brittle shotcrete and made the Administration ban steel fibres in their contracts for nearly ten years.

4 THE EIGHTIES

4.1 Research

Tests on steel fibre reinforced shotcrete in the large test rig described above indicated that such a shotcrete might exhibit a plastic behaviour in shear in the cracked stage. Small scale shear tests were performed and they showed that steel fibre reinforced shotcrete displays very good deformation capacity in shear and that it exhibits plastic characteristics with a load-deformation curve that does not end until most fibres are torn out (Holmgren, 1985a).

Questions about the design of shelters and military caverns situated in rock near the surface were raised regularly—especially since steel fibres had become the dominant reinforcement method for shotcrete. A research project was undertaken in order to find out the optimum design of such facilities. In this project a large test series on dynamically loaded unreinforced and reinforced shotcreted specimens was performed (Holmgren, 1985b, Holmgren & Ansell, 2006). The project resulted in some important principles for the design of dynamically loaded shotcrete linings. Dynamic loading from rock blocks excited by bomb detonations, etc., causes primarily a bond failure at the interface between the rock and the shotcrete. The loosened shotcrete itself has a low energy absorbing capacity, so the momentum from the rock blocks must be absorbed by rock bolts with sufficient deformation capacity. The reinforced shotcrete layer shall thus be designed for a distributed load which corresponds to the yield load of the bolts. That load shall be transferred to the rock bolts by a device that has a larger load capacity than the bolt in order to avoid a localized, brittle failure. The shotcrete layer can be designed using yield line calculations where the residual bending strength is used as the yield capacity.

4.2 Design

Steel fibre reinforced concrete was developed by materials researchers who were not focused on the need of the structural engineer to have a strength estimate for determining the dimensions of a structure. For steel fibre reinforced shotcrete in actual applications where yield line calculations were performed, knowledge about the yield strength of the fibre concrete was needed. It was also quite clear

that steel fibre reinforced concrete could not be defined in contracts by type and amount of fibres, but a requirement should be based on the capacity demanded by the structure. For Swedish contracts a requirement was formulated based on the load-deflection curve for a notched beam (the JH method), (Holmgren, 1985c). The requirement was very simple to use for the assessment of the actual material. The beam curve should pass over a certain minimum level and fall down beyond a certain level of deformation. The level was chosen so that shotcrete with inferior tensile bending strength was excluded and the position of the point was chosen so that a shotcrete with inferior residual strength was excluded. See Figure 2.

This concept is very similar to the one now introduced in Eurocodes.

4.3 Practice

The wet mix method had now become the dominating spraying method in Sweden. On the fibre market the Bekaert zl-fibres and the EE fibres were the most widely used. The contracting company Besab experimented with fibre addition using a specially designed fibre feeder that was able to keep the fibres apart and deliver them to a specially designed nozzle, where concrete and compressed air were added. The fibre feeder required one extra operator and became thus too expensive.

Another contractor, Ekebro, experimented with a device that cut fibres from a cold drawn wire during the spraying process. The fibres had to be delivered with a very high speed to the nozzle, which caused unwanted stops too often. This equipment was used in a factory where balcony facades

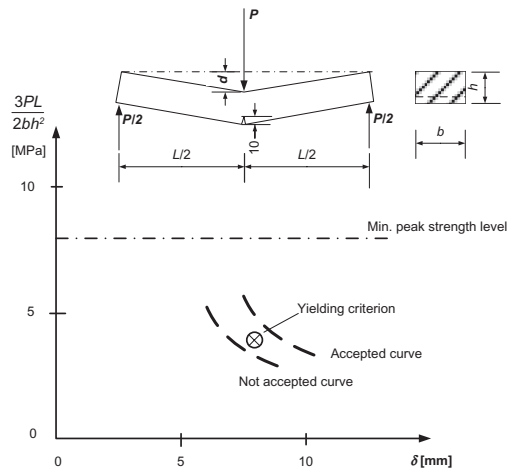


Figure 2. Early Swedish method for defining requirements for steel fibre reinforced shotcrete (the JH method).

were produced, but it was never introduced into production underground. The Bekaert fibres and the EE fibres could easily be added to the mixed concrete, which gave them a market advantage.

The need for a handbook for testing of shotcrete properties was strongly felt during this decade and one was written that contained advice concerning the spraying of test boxes, wash out testing for the determination of fibre content and performance of beam tests (Alema et al, 1986). Beam tests according to ASTM C-1018 and the determination of toughness indices were described as well as the JH method.

5 THE NINETIES

5.1 Research

This decade was dominated by problems connected with shotcrete drains. The old types based on mineral wool and plastic sheets did not perform well. Since large infrastructure projects were planned in Sweden an improved shotcrete drain was absolutely necessary to develop. In a large R&D project the Swedish Road Administration investigated and evaluated a number of designs with reference to constructability, behaviour at freezing, fatigue and possibility to flush them clean inside, (Fredriksson et al, 1996). The basic design consisted of an insulation sheet with closed pores which is covered with steel fibre reinforced shotcrete fixed to the rock surface on both sides of the insulation sheet by bond only. See Figure 3.

This drain concept was used in a Swedish railroad tunnel project before the final report by Fredriksson et al was published and it was subjected to measurements of air pressure variations and accelerations, (Holmgren, 1994) and (Bäck & Oscarsson, 1995). In 1996 I got the chair of Concrete Structures which gave me a possibility to employ PhD students and focus a little more on shotcrete problems.

One important issue in all tunneling projects is the influence of vibrations from blasting on fresh shotcrete. This was studied in one part of a thesis from the division of Concrete Structures by (Ansell, 1999). One interesting result was that thicker linings are more sensitive to vibrations than thinner ones. This research is still going on with a PhD student under supervision by Ansell.

In 1998 I got a Swedish patent on a yielding rock bolt which was intended to be used as the energy absorbing element in a dynamically loaded shotcrete lining. The bolt was thoroughly studied both experimentally and theoretically by Ansell in one part of his doctoral thesis. The rock bolt is used by the Swedish Defense in some caverns together with steel fibre reinforced shotcrete.

5.2 Design

The ASTM C-1018 standard for requirements on shotcrete was adopted. It could not be applied on our contracts in Sweden since it did not make it possible to define design strength of the steel fibre reinforced shotcrete from the toughness parameters I_n . After some communication between Colin Johnston, Åke Skarendahl and me the residual strength parameters $R_{m,n}$ were introduced. By multiplying this parameter by the strength at first crack it was possible to determine a residual strength, which could be used for yield line calculations.

In 1992 a handbook on the design of shotcrete linings in hard rock was published by the Swedish State Power Board, (Holmgren, 1992). It was also accepted as a design guide by the Swedish Road Administration and the Swedish Railroad Administration for several years before they had written their own codes for tunneling. The handbook contained recommendations regarding the design of shotcrete linings for different types of cracked rock and diagrams for the calculation of the dimensions of the lining and rock bolting for both static and dynamic loading. A special chapter was written about the handling of errors. I had observed many times that owners often were badly prepared when requirements were not met. In such situations it is very important to act in a rational way, so that the required actions are regarded as logical in relationship to the original requirements. Otherwise there will be an unnecessary debate with the contractor and perhaps even suspicions that the owner wants to punish the contractor. According to my experience there is seldom a problem in convincing the contractor that he has to take the responsibility for his mistakes if you can explain why the original requirements are in place and why additional actions are necessary to restore the functionality of the lining.

During the nineties a more or less standardized design for traffic tunnels developed. Those tunnels are always fully sprayed with bonding, steel fibre reinforced shotcrete in the lowest reinforcement class. No unreinforced shotcrete or unlined walls were permitted anymore! In the next class the shotcrete is a bit thicker and is combined with rock bolts with washers outside the shotcrete layer. The third class is similar to the second class but the shotcrete layer is thicker and the bolts more closely spaced. A fourth class often consists of an arch solution to be applied where rock anchors cannot be used, e.g. where the overburden is small.

5.3 Practice

The handbook on testing and assessment of steel fibre reinforced shotcrete was revised and

rewritten, (Holmgren et al, 1997). This edition was more directed towards rock reinforcement than the previous one. References to actual standards were introduced as well as the modified ASTM C-1018. The JH method was now abandoned since the modified ASTM method allowed the determination of a residual strength value, which could be used in yield line calculations.

The contractors had by now understood that the owners do not accept mistakes in the control procedures. Bond testing as well as sawing out test cubes and beams and testing them are now performed by specialist companies. The most problematic requirement is the one on residual strength. Because of the large scatter in this parameter one beam test often falls well below the requirement. Other requirements are normally met.

6 YEAR 2000 AND FORWARD

6.1 Research

In a PhD project which was started before 2000, the mode of action of bolt anchored steel fibre reinforced shotcrete was studied, (Nilsson, 2000 & 2003). Nilsson showed both experimentally and by FEM calculations how the punching failure in steel fibre reinforced shotcrete in the vicinity of a washer develops. He also explained why the increase in load carrying capacity because of dome action from restrained horizontal deformation of a shotcrete slab was much larger than was expected from theories and experience from testing of conventionally reinforced concrete slabs.

This period was otherwise dominated by shrinkage and cracking. When the large traffic tunnel system Södra Länken south of Stockholm was constructed there were many complaints about extensive cracking in the bonding shotcrete. It was assumed that the water curing was not well performed. The cracking also seemed to decrease when special teams were set up to implement water curing.

When the tunnels were finished it was discovered that a large number of drains, especially the continuous ones were heavily cracked. Also in the large railway project in northern Sweden, Botniabanan, it was felt that the cracking problems were more serious than normally occurs. Suspicions soon were directed towards the alkali free accelerators that were introduced approximately at the same time as the increased frequency of cracking was observed. Everybody knows that water curing of shotcrete seldom is performed sufficiently, but this was different from ordinary experiences. Perhaps shotcrete with alkali free accelerators was more water demanding and more prone to shrinkage.

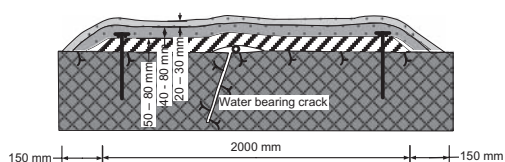


Figure 3. The Swedish drain consists of a perforated plastic tube at the water bearing crack, which is covered by an insulating sheet with closed pores. The sheet is fixed to the rock by short bolts and reinforcement bars. The sheet is covered with steel fibre reinforced shotcrete which bonds to the rock surface at the edges. Finally the steel fibre reinforced shotcrete is covered by unreinforced shotcrete.

A project was started where the structure and shrinkage of differently accelerated shotcretes were studied by Prof. Björn Lagerblad and his team at the Swedish Cement and Concrete Institute and the shrinkage cracking problem was studied by Dr Ansell and me.

The first project showed that ettringite crystals that are created by the alkali free accelerators form an open structure that gives a larger water loss than in ordinary concrete. It was also shown that its shrinkage was larger than experienced by ordinary shotcrete.

With the commonly used cement for shotcrete in Sweden (CEM I, MH, LA, SR) and normal rock temperature it will take more than 24 hours before the shotcrete will get any significant strength. In normal concrete the cement reactions occur in a plastic structure which means that the chemical shrinkage can take place without causing any problems. In accelerated shotcrete they occur in an already existing, "stiff" structure consisting of ettringite crystals why the chemical shrinkage will affect micro cracking. An effect of this is an autogeneous shrinkage of around 0.5 mm/m. Thus water added from the start is needed. Moreover, the hardened shotcrete will get a coarser pore structure, which will increase the drying shrinkage by between 15 and 20%. Together this produces shrinkage levels that can explain the increased cracking observed in the field.

The amount of shrinkage is dependent on the amount of cement paste in the mix. Normally shotcrete contains more cement (and water) than ordinary concrete. By careful proportioning with increased amounts of filler it is possible to reduce the amount of cement by around 20%, but one will get problems at low w/c ratios. One effective method is, however, to add substantial amount of air to the mix, air that disappears during the shooting operation. Another effective method is to add shrinkage reducers to the mix (Lagerblad et al, 2007).

In the shrinkage cracking project some preliminary tests with shrinkage rings were performed. The concrete used was a shotcrete mixture which was cast instead of sprayed. The tests showed that normal amounts of steel fibres cannot prevent cracking but if glass fibres are added the combination of fibres efficiently prevents the formation of micro cracks so that larger cracks never form. The explanation is probably that shrinkage stresses successively relax while the glass fibres keep the micro cracks together since shrinkage is a slow process. When beam specimens with steel fibres and glass fibres were loaded under short term conditions they all cracked in a way that is typical for a strain softening material and it did not appear as if the glass fibres made any difference at all, (Ansell & Holmgren, 2007a; Holmgren & Ansell, 2008).

The research by Ansell on young shotcrete subjected to vibrations from blasting is continuing (Ansell, 2004, 2007b). A FE-model has been developed and is now being refined. A PhD student is also working in this project since 2008.

6.2 Design

The introduction of high-speed trains focused attention on the design of shotcrete drains. The Swedish Railroad administration increased the design loads for those structures on at least two occasions, so now shotcrete drains have to be designed for an air pressure ranging from ± 3 kPa at single track tunnels and speeds 170–219 km/h up to ± 5 kPa at double track tunnels and speeds 220–270 km/h. The design shall also take fatigue into consideration.

In the standard design the drain is fixed to the rock by bond only, but also other concepts exist where anchoring devices are sprayed in.

The drain must either be designed assuming that a shrinkage crack has formed at an unfavourable position or a crack control device shall be installed so the position of a shrinkage crack is known in advance.

6.3 Practice

Shotcrete drains have continuously caused practical problems during this decade. The first problem has been related to shrinkage cracking. Such cracks have of course raised the question whether the cracks have decreased the load carrying capacity or not. A tremendous amount of work has been performed mapping cracks in drains. Many of the cracks are parallel to the direction of the tunnel and thus more or less harmless. Others are perpendicular to the tunnel direction but very limited in length and thus considered harmless. Some are both directed perpendicularly to the tunnel direction and of several meters length and thus regarded to be a potential danger.

In the last case the drain capacity was recalculated taking the crack into consideration and if the thickness was at least equal to the nominal amount some of them were accepted because the shotcrete strength increased considerably by that time. In the opposite case the drain has been made thicker by additional spraying.

The second problem is connected to the first one and that has to do with the spraying technique. Nozzlemen have not been made aware that there is a difference between shotcrete on the rock surface and shotcrete on a drain. In the latter case the shotcrete is designed to carry a comparatively large load and thus the thickness is of great importance.

Traditionally the thickness of shotcrete is measured according to a standardized pattern. That pattern does not take the function of a drain into consideration. In a drain the thickness is most important where the bending moments are largest but it has also to be considered that the thickness shall be constant over the drain in order to minimize the risk that shrinkage cracks are concentrated at the weak parts.

7 CONCLUSIONS

It has been demanding and challenging to work with problems connected with shotcrete in tunnels. The demands come from the fact that tunneling people are very pragmatic and that many of them are very experienced. The research must therefore give answers to questions that are regarded as realistic by people involved in shotcreting practice.

The challenge comes from the fact that rock problems are so complex that they seldom can be described with any accuracy. Yet the solutions must be possible to adopt in varying practical situations. It is also to be remembered that the conditions in a tunnel are such that actions which seem to be simple and clear on a drawing or in a description, might not even be possible to perform.

I am very happy that chance once lead me into this field of technology and that I have been allowed not only to contribute to the development of engineering technology but also to solutions to practical problems.

REFERENCES

- Alemo, J., Holmgren, B.J. & Skarendahl, Å. 1986. Stålfiberbetong—provning och värdering. (Steel fibre concrete—testing and assessment. *In Swedish*.). Stockholm: Bygghörlaget.
- Ansell, A. 1999. Dynamically Loaded Rock Reinforcement. Doctoral Thesis, TRITA-BKN. Bulletin 52, 1999, ISSN 1103-4270, ISRN KTH/BKN/B-52-SE.

- Ansell, A. 2004. A finite element model for dynamic analysis of shotcrete on rock subjected to blast induced vibrations. Proc. 2nd International Conference on Engineering Developments in Shotcrete, 4 October, Cairns, Queensland, Australia. *Shotcrete: More Engineering Developments*. Leiden: Balkema.
- Ansell, A. & Holmgren, B.J. 2007a. Sprutbetongs krympning—fiberinblandning för bättre sprickfördelning. (The shrinkage of shotcrete—fibre mixes for better crack distribution. *In Swedish*). SveBeFo rapport 87, Stockholm 2007, ISSN 1104-1773, ISRN SVEBEFO-R-87-SE.
- Ansell, A. 2007b. Shotcrete on rock exposed to large scale blasting. *Magazine of Concrete Research*, 59(9), 663–671.
- ASTM C-1018, 1992. Standard test method for flexural toughness and first-crack strength of fibre-reinforced concrete (using beam with third-point loading), *Annual book of ASTM Standards*, Vol 04.02 Concrete and Aggregates, ASTM, Philadelphia, USA.
- Bäck, L. & Oscarsson, J. 1995. Fältundersökning av sprutbetongdräner i tunnlar utsatta för lufttrycksvariationer vid tågpassage. (Field investigation of shotcrete drains in tunnels subjected to air pressure variations from passing trains. *In Swedish*). TRITA-BKN. Examensarbete 51, Byggnadsstatik 1995, ISSN 1103-4297, ISRN KTH/BKN/EX-51-SE.
- Fredriksson, A. et al. 1996. Dräner för bergtunnlar—funktion och utformning. Ringen och yttre tvärleden, Projekt nr 320. (Drains for tunnels in rock—function and design.... *In Swedish*).
- Hahn, T. & Holmgren, B.J. 1979. Adhesion of shotcrete to various types of rock surfaces and its influence on the strengthening function of shotcrete when applied on hard jointed rock, 1979, *Proc. 4th Int. Congress on Rock Mechanics*, Vol. 1, pp. 431–440, International Society for Rock Mechanics, Montreux, Suisse.
- Holmgren, B.J. 1979. Punch loaded shotcrete linings on hard rock. Rapp nr 121:6, FortF/F dnr 2492F, Stockholm.
- Holmgren, B.J. 1985a. Bolt anchored steel fibre reinforced shotcrete linings. Stiftelsen Bergteknisk Forskning 73:1/85, Stockholm.
- Holmgren, B.J. 1985b. Dynamiskt belastad bergförstärkning av sprutbetong. (Dynamically loaded shotcrete linings. *In Swedish*). Fortifikationsförvaltningen, forskningsbyrån, rapp A4:85, Stockholm.
- Holmgren, B.J. 1985c. Advanced tunnel support using steel fibre reinforced shotcrete. *Steel fiber concrete—US-Sweden joint seminar (NSF-STU) Stockholm, June 3–5, 1985*. London: Elsevier.
- Holmgren, B.J. 1992. Handbok i bergförstärkning med sprutbetong. (Handbook in rock reinforcement with shotcrete. *In Swedish*). Stockholm: Vattenfall Vattenkraft.
- Holmgren, B.J. 1994. Dimensionering av sprutbetongdräner i tunnlar utsatta för lufttrycksvariationer vid tågpassage. (Design of shotcrete drains in tunnels subjected to air pressure variations from passing trains. *In Swedish*). Investigation for the Swedish Rail Administration, Stockholm.
- Holmgren, B.J., Alemo, J. & Skarendahl, Å. 1997. Stålfiberbetong för bergförstärkning. Provning och värdering. (Steel fibre reinforced shotcrete for tunnel linings. *In Swedish*). CBI-rapport 3:97.
- Holmgren, B.J. & Ansell, A. 2006. Design of Bolt Anchored Reinforced Shotcrete Linings Subjected to Impact Loadings, *Shotcrete for Underground Support X, September 12–16, 2006, Delta Whistler Resort, Whistler, BC, Canada*, ISBN-13: 978-0-7844-0885-8.
- Holmgren, B.J. & Ansell, A. 2008. Shrinkage of Shotcrete—Fibre Mixes for Better Crack Distribution. 5th International Symposium on Sprayed Concrete. Lillehammer, Norway 21–24 April 2008. *Modern Use of Wet Mix Sprayed Concrete for Underground Support*. ISBN: 978-82-8208-005-7.
- Lagerblad, B., Fjällberg, L. & Westerholm, M. 2007. Sprutbetongs krympning—modifiering av betongsammansättning. (Shrinkage of shotcrete—modification of the concrete mix. *In Swedish*). SveBeFo rapport 86, Stockholm 2007, ISSN 1104-1773, ISRN SVEBEFO-R-86-SE.
- Nilsson, U. 2000. Load bearing capacity of steel fibre reinforced shotcrete linings. Licentiate Thesis, TRITA-BKN. Bulletin 56, 2000, ISSN 1103-4270, ISRN KTH/BKN/B-56-SE.
- Nilsson, U. 2003. Structural behaviour of fibre reinforced sprayed concrete anchored in rock. Doctoral Thesis, TRITA-BKN. Bulletin 71, 2003, ISSN 1103-4270, ISRN KTH/BKN/B-71-SE.

Design and construction of a permanent shotcrete lining— The A3 Hindhead Project, UK

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ABSTRACT: This paper discusses the design and construction of a permanent shotcrete lining for the 1.8 km long A3 Hindhead twin bore highway tunnel that commenced construction in January 2008 and for which excavation and support was completed in March 2009. The tunnel is excavated within the Hythe Beds—a 90 m thick sequence within the Lower Greensand Series formation consisting of ‘weak, to very weak sandstone with occasional thin beds of fine sand’. The proposed construction methodology was progressive mechanical excavation of a horseshoe shaped tunnel with shotcrete installed in each excavation cycle using robotic spraying equipment. This paper describes the development of the innovative ‘permanent’ primary lining design including the modelling of the lining, specification of a durable shotcrete mix including early strength requirements, as well as details of the construction. This construction technique has proved to be very successful in ‘soft’ rock, and has local applicability to Auckland’s bedrock material, East Coast Bays Formation.

1 INTRODUCTION

The A3 Hindhead project located in Surrey is the UK’s latest bored highway tunnel. The development of the design has incorporated recent advances in tunnelling technology resulting in an innovative approach to many aspects of the tunnel support systems, and in particular the shotcrete lining.

The project is a 6.7 km dual carriageway trunk road that includes a 1.83 km tunnel being delivered under a UK Highways Agency Early Contractor Involvement (ECI) contract. The tunnel has been constructed using a sequential excavation method whereby shotcrete was sprayed at the face following each advance by a tunnel excavator, also known as Sprayed Concrete Lining (SCL) in the UK and as the New Austrian Tunnelling Method (NATM) in Europe.

This paper describes the development of the innovative ‘permanent’ primary lining design including: the modelling of the lining; specification of a durable shotcrete mix including early strength requirements; and testing and construction requirements. The use of a sprayed applied membrane has also enabled the use of a sprayed shotcrete secondary lining. This construction technique has proved to be very successful in ‘soft’ rock, and has local applicability to Auckland’s bedrock material, East Coast Bays Formation.

2 BACKGROUND

The A3 Hindhead project is one of the schemes in the UK Government’s Targeted Programme of Trunk Road Improvements. The project will complete the dual carriageway link between London and Portsmouth and remove a major source of congestion, particularly around the A3/A287 traffic signal controlled crossroads. Refer to Figure 1 for location details.

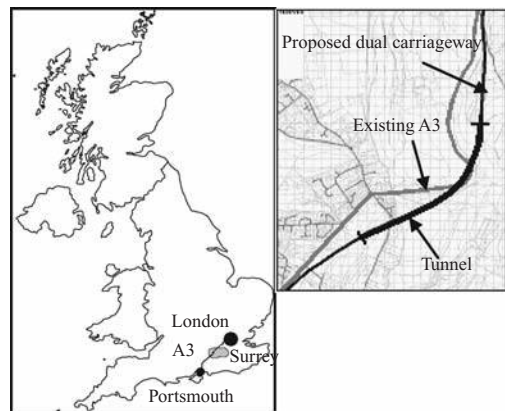


Figure 1. Project location.

The project will deliver quicker, more reliable journeys on a safer road, and remove much of the present peak time “rat-running” traffic from unsuitable country roads around Hindhead. The centre of Hindhead will be freed from the daily gridlock that blights the area, with the result that the project will bring benefits to road users, local residents, and the highly prized local environment. Construction of the project commenced in January 2007, and the tunnelling commence in January 2008, with excavation and support completed in March 2009. The tunnel is planned to be opened in 2011, with the scheme completed in March 2012.

2.1 Early Contractor Involvement (ECI) Contract

The Highways Agency (HA) introduced the principle of “Early Contractor Involvement” in 2001. This new form of procurement is concerned with bringing suppliers and designers together much earlier in scheme conception than previously occurred, allowing them to work together more closely. This allows more scope for innovation, improved risk management, better forward planning of resource requirements and minimisation of long term environmental impacts, improved consideration of buildability and health and safety, shorter construction periods and reduced environmental impacts during construction.

Overall, the HA considers that the early creation of delivery teams offers the opportunity for better value and improved performance.

3 GEOLOGY

3.1 Overview

The geology of the Hindhead area comprises a sequence of fine grained sedimentary deposits laid down during the Lower Cretaceous period in near shore transgressive marine conditions on the margins of the subsiding Weald Basin. The tunnel is within the Hythe Beds—a 90 m thick sequence within the Lower Greensand Series formation.

The Hythe beds are variably sorted, highly glauconitic, variably bioturbated and cross-bedded sands and sandstones. The Hythe bed unit is divided into six litho-stratigraphic subdivisions, through four of which the tunnel passes.

3.2 Tunnelling conditions

The tunnel at the southern end passes through units Upper Hythe A and B which are similar units with an increasing number of sandstone bands with

depth, described as ‘medium dense thinly bedded and thinly laminated, clean to silty and clayey fine and medium sand with subordinate weak to strong sandstone, cherty sandstone and chert’.

The majority of the tunnel passes through the more competent Upper Hythe C and D, and Lower Hythe A units, described as ‘*Weak, locally very weak to moderately strong, slightly clayey fine to medium SANDSTONE with occasional thin beds of clayey/silty fine sand*’. The remaining unit is Lower Hythe B which has been avoided by the tunnel as clays and sand become dominant in the lower half of the unit.

The sandstone within Upper Hythe C/D and Lower Hythe A has typical UCS values of between 2 and 5 MPa and is heavily fractured with six joint sets including the sub-horizontal bedding with mean fracture centres varying between 190 and 815 mm. The tunnel also intersects a small number of discrete thin bentonitic Fuller’s Earth Beds, formed from air- and water-borne volcanic ash, including crystal and lithic tuffs. The tunnel is above the historically observed water table, with the maximum predicted water table exceeding the invert level in only one location by a depth of less than 1 m. Refer to Figure 2 for a geological longsection.

3.3 Ground behavior model

A challenge for this project was to define a ground behavior model in an unusual material that had not been tunnelled previously. The difficulties in interpretation stem from the weak to very weak nature of the sandstone material in combination with the content of up to 20% interbedded soil layers. A recurring challenge in tunnelling is to determine the rock mass strength and stiffness, with empirical methods such as RMR, Q-method (Bieniawski, 1984), or GSI (Marinos and Hoek, 2000) often used. The strength and stiffness relationships that are the cornerstone of these methods are generally determined from data for significantly stronger rocks than the 2–5 MPa sandstone and do not take account of the influence of the soil layers leading to a significant over-estimate of stiffness.

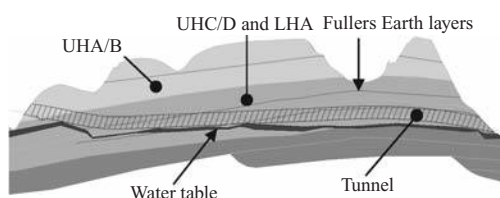


Figure 2. Geological longsection.

An extensive geotechnical investigation was undertaken with sonic testing, pressuremeters and triaxial testing all used to determine the Elastic Modulus of the rock mass. Pressuremeter testing was found to be the most reliable with the sonic testing over-estimating the stiffness, and the triaxial testing surprisingly under-estimating the stiffness in a number of cases. This was thought due to the difficulty in finding a 300 mm long specimen for testing in a material with 6 joint sets and an average bedding spacing of 190 mm. Fortunately some good quality rock joint shear box testing was undertaken that provided a lower bound for strength and stiffness interpretations. The interpreted model used in design was a small strain stiffness model (E_{ea}) that varied the stiffness of the rock mass with strain, and a Mohr-Coulomb strain softening model used for strength.

4 TUNNEL CROSS SECTION

The Hindhead tunnel layout comprises twin 2-Lane bores with cross passages at 100 m nominal centres. Refer to the typical cross section in Figure 3 below. Each bore has two 3.65 m lanes, with full batter curbs and 1.2 m wide verges on each side of the tunnel. The verge width is sufficient to allow for sight-lines due to the horizontal curvature of the tunnel, to accommodate electrical services and also to provide wheelchair access to the cross passages and emergency points at 100 m nominal centres along the tunnel. The vertical traffic gauge provided is 5.03 m with an additional clearance of 250 mm to the Equipment Gauge to allow for flapping tarpaulins and other transitory gauge infringements.

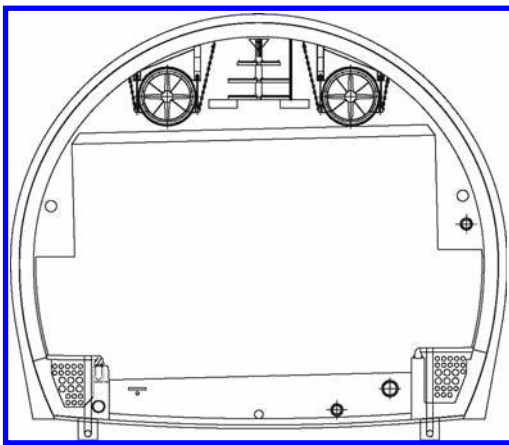


Figure 3. Typical tunnel cross section.

5 TUNNEL EXCAVATION AND SUPPORT TYPES

The presence of the sand layers, in one location up to 2 m thick, led to the selection of a sequential excavation method whereby Fibre Reinforced Shotcrete (FRS) is sprayed at the face following each excavation advance, also known as Sprayed Concrete Lining (SCL) in the UK and as the New Austrian Tunnelling Method (NATM) in Europe. Standard hard rock tunnel support techniques such as pattern bolting were not considered suitable due to the sand layers and the very low bond stress negatively impacting the effectiveness of rock reinforcement.

Four basic support types were designed for the standard tunnel cross sections with minor variations required at cross passage junctions and Emergency Point niches. There is one main support type for the sandstone section, with three support types covering the section through sand and the transition from sand to sandstone.

Excavation and support types were specified based on tunnel chainage and have been designed to cover all expected ground conditions. It was not proposed that support types be selected based on geological inspection. The Hythe beds had 6 joint sets and an average joint spacing of less than 200 mm. The UCS values were typically 2–5 MPa. This material was expected to act as a continuum, and given the heavily fractured nature of the material, and presence of sand layers, meaningful variations in rock quality were expected to be difficult to detect. A suite of ‘additional measures’ discussed below were designed to account for any local stability issues. Geological inspection and mapping of the open excavation, and monitoring results, were used to determine the advance length that may have varied between 1 m and 2 m, with 1 m advances specified at critical locations such as beneath surface structures and roads.

5.1 Support Type 1

At the northern end of the tunnel (Chainage 3120 to 4650), excavation was in rock (UHC/D, LHA) and Support Type 1 was specified throughout. The tunnel was generally excavated with a full face heading followed at a distance by the bench excavation. Due to the generally stable nature of the ground and tunnel location above the water table, a closed invert was not required and the horse-shoe shaped primary lining was supported on elephant’s feet. Refer to Figure 4 for details of the primary lining in the sandstone section.

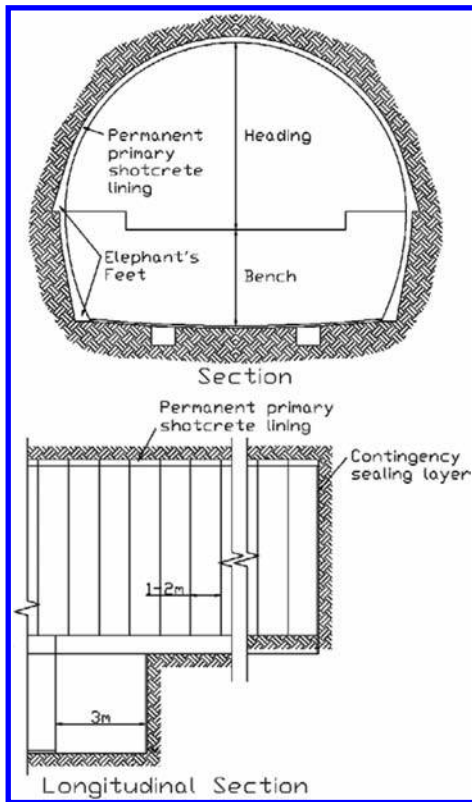


Figure 4. Support Type 1 in the SANDSTONE section.

5.1.1 Additional support measures

In addition to the sequences and support requirements for each of the support types, contingency 'additional support measures' were specified. The requirement for the contingency measures was triggered by geological inspection and mapping, and monitoring results, and included:

Spiling. Self drilling GRP tubular spiles were installed when ground conditions resulted in excessive overbreak, or instability in the crown. Spiling was detailed as mandatory for some of the Support Type 1 excavation, in areas of known potential crown overbreak such as where the tunnel crosses Fullers Earth bands, and where a 2 m thick layer of sand and shattered rock intersected the crown. During construction it was found that there were less areas of instability and only 26% of the tunnel required forward support compared to the initially anticipated 39%.

Additional face support measures. Geological inspections informed the selection of additional face support measures such as sealing layers, face support by the later excavation of a central support wedge, or face dowels.

Probe Drilling. Probe holes drilled ahead of the face were specified for the entire length of Support Type 1 to relieve any potential hydrostatic pressure from perched water ahead of the face. If water was detected in the probe holes then additional holes were drilled to drain any water.

Grout Stabilisation. Microfine cement or chemical grouts were specified to stabilise running sand bands, or other local areas of instability caused by sandy layers. In the event this was not required.

Invert strut. Convergence monitoring was undertaken during construction with a three stage trigger limit system implemented. Unplanned convergence resulting from worse than expected ground conditions was addressed by installation of an invert strut at bench level.

5.2 Support Types 2-4

At the South end of the tunnel (Chainage 2880 to 3120 (m001)), excavation was in sand (UHA/B) and support types 2, 3 and 4 were specified. Refer to Figure 5 below for details. The excavation was

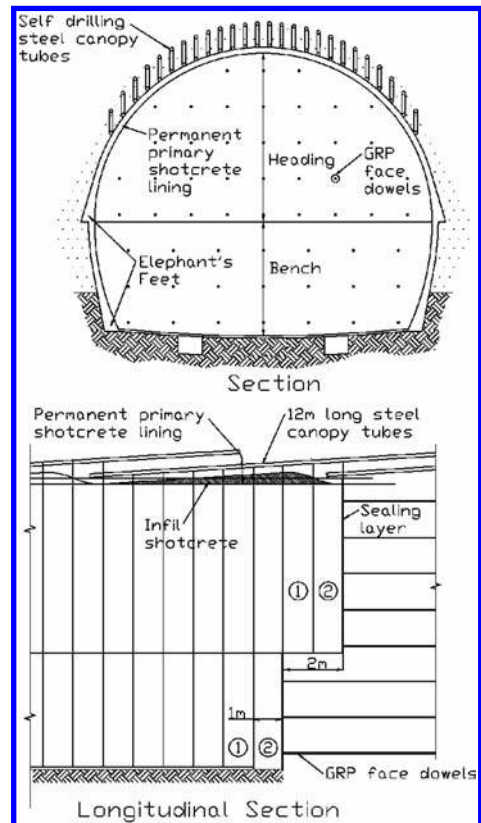


Figure 5. Support Type 3 which is typical for the SAND section.

carried out on dayshift only due to constraints on working hours and was made stable with the use of a steel pipe umbrella and face dowels. The pipe canopy comprised 12 m long 139.7 mm diameter tubes at 425 mm centers with an overlap of 4 m. The advance length was a maximum of 1 m for these support types.

Support Type 2 had sandy material (UHA/B) in the heading only, with the heading elephant's feet supported on the sandstone material (UHC/D). This meant GRP face dowels were required in the heading only, and the heading could advance ahead of the bench. The face dowels were 12 m long with a 4 m overlap and were installed with the same drill jumbo used to install the pipe canopy.

Support Type 3 had a full face of sandy material with the elephant's feet of the bench supported on the sandstone material. As the heading elephant's feet were not supported on sound material, the heading was advanced with the bench, with a 2 m separation provided to maintain face stability. Face dowels were required for both the heading and the bench. Support Type 4 had a full face of sandy material that extended below the tunnel, and therefore a closed invert was required. The heading had to be advanced with the bench, with the invert closed at a maximum of 6 m behind the face.

6 DESIGN APPROACH AND METHODOLOGY

The excavation sequences outlined above were designed to control strains in the ground so that as much as possible of the ground load bearing capacity was used and the strains maintained at levels that minimised yielding.

6.1 Analysis

The main design approach was to utilise numerical methods of analysis (FLAC Version 5.0) to model in 2-D the different excavation stages and predict the performance of the linings in terms of the stresses and corresponding deformations. The model included the non-linear small strain stiffness material model and Mohr-Coulomb failure criterion with strain softening to residual strength parameters. The models included the insitu stress regime with the horizontal stress modelled between 0.7 and 1.15 of the vertical stress, with 0.5 also used in the lower cover regions.

A modelling simplification inherent in 2-D modelling is the relaxation of the ground ahead of an excavation face, a largely 3-D effect, and the time-dependent development of lining strength and stiffness. In order to determine the appropriate

value for these ground conditions a 3-D numerical model (FLAC Version 3.0) was utilised, which can accurately model the ground relaxation based on the proposed construction sequence and also the time-dependent strength and stiffness development of the shotcrete. For the 2-D model, the 28 day stiffness value was used in accordance with Eurocode 2, divided by a creep and shrinkage factor of 2. This factor reduces the stiffness of the sprayed concrete lining to take account of the creep and relaxation that occurs when loading during early age (John and Mattle, 2003). The 2-D model was then calibrated against the 3-D modelling to determine an equivalent relaxation of 60%. In comparing the 2-D analysis with the 3-D analysis it was found that moments were over-estimated in 2-D at the heading stage and slightly under-estimated at full excavation stage. Axial loads were consistently over-estimated in 2-D everywhere except at the elephant's feet, however this was due to the increase in mesh fineness at the elephant's feet locations used in the 2-D analysis.

In addition to providing calibration of the relaxation figure, the 3-D FLAC modelling was also used to model stability of the face, early age capacity of sprayed concrete lining near the face, and the effect of variations in advance length.

6.1.1 Shotcrete strength gain

There are various different models on which to base the strength gain properties of the sprayed concrete lining. The J2 and J3 curves from the Austrian Sprayed Concrete curves (Osterreichischer Betonverein, 1999), and an alternative model for tunnel shotcrete (Chang & Stille, 1993), where the properties (Y) of young shotcrete are estimated from 28 day values (Y_{28}) with the expression:

$$Y = a \cdot Y_{28} \cdot e^{(ct)^{0.7}} \quad (1)$$

where a and c are constants that vary for each property (values for which are provided in the paper); and t = time in days. This equation indicates lower shotcrete strengths than the J curves during the first 3 hours of shotcrete life, but provides a strength gain curve for longer than the first 24 hours of the concrete's behaviour.

For the A3 Hindhead project tunnel shotcrete, an upper J2 curve which is half way between the J2 and J3 curves, was specified. In order to test the sensitivity of the design to the strength gain, modelling was undertaken using both the Chang & Stille shotcrete strength model and the Upper J2 curve up until 24 hours followed by the Chang & Stille relationship. For details of the differences between the two strength models refer to Figure 6.

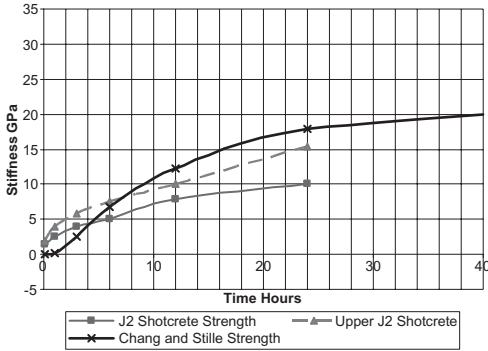


Figure 6. Shotcrete stiffness vs time relationships.

The stiffness of the concrete was also based upon an equation from Chang & Stille:

$$E = 3.86 \sigma_c^{0.60} \quad (2)$$

where σ_c = compressive strength of the concrete at the point in time at which the concrete stiffness is being calculated. Stiffness develops at a faster rate than strength, which can result in a high loading rate on the shotcrete. Eurocode 2 defines the concrete stiffness as:

$$E_{cm} \text{ (GPa)} = 22[(f_{cm})/10]^{0.3} \quad (3)$$

where f_{cm} = mean compressive strength of the concrete at 28 days. This is then reduced by 10% due to the inclusion of Limestone in the concrete aggregate. In order to model how the stiffness of the concrete changes with age the equation used is:

$$E_{cm}(t) = (f_{cm}(t) / f_{cm})^{0.3} E_{cm} \quad (4)$$

The Eurocode equation does not allow for as much creep of the young concrete, which is important in the behaviour of shotcrete. Modelling was undertaken using the Chang & Stille equation as it was proposed particularly with sprayed concrete in mind. The Chang & Stille stiffness relationship may still not be entirely suitable for very early age shotcrete and is probably still an over-estimation of stiffness. In order not to design over-conservatively for very young concrete the ULS has not been used for concrete less than 2 days old, and a reduced factor of safety was considered acceptable.

The analysis indicated that for the heading excavation the Upper J2 strength curve, which gains strength and stiffness fastest, attracts more axial load. The Chang & Stille model starts as the weakest so has the lowest axial load at the lining. The moments are generally higher for Upper J2 but the Chang & Stille model results in the peak moment

due to greater deflections resulting from a lower stiffness lining.

The choice of strength gain curve had the greatest impact for the shotcrete advances just behind the face. If the strength of the concrete does not increase quickly enough the excavation rate may have to slow. Having a rate fast enough is particularly important at the south portal where work can only take place during the day. For this reason, despite indications that the lower stiffness at early age of the J2 curve results in lower final axial loads and moments in the lining, it does not gain strength quickly enough to progress at a fast enough pace at the face. In the event during construction the contractor elected to implement a 3 m exclusion zone at the face, and as the early strength was not as critical the J2 strength gain curve could be used thereby saving the need for the additional additives required to achieve the higher strengths at very early age.

6.1.2 Heading advance length analysis

In order to evaluate the effect of advance length on load development in the shotcrete lining, analyses were carried out with a variation in excavation step size of 1, 1.5 and 2 metres.

For the 1.0 m heading excavation step, the loads on the lining were found to have a short term safety factor of around 1.4 at 3 m back from the lining face. This meant that the span that needed to be self-supporting was 4 m. Using typical stand-up time curves against RMR (Bieniawski, 1984) which plots roof span against stand-up time in hours, a rock mass rating (RMR) of 40 or greater would be needed. This meant that the tunnel would be self supporting for the 32 hours required for the concrete to gain enough strength to provide support, assuming 4 hours per excavation step with 50% contingency. If the RMR was found to be less than 40 spiles should be inserted to add support to the excavation. For the 1.5 m and 2.0 m heading excavation steps, to achieve the equivalent safety factor, a rock mass rating (RMR) of 45 and 50, respectively, or greater, was required for these advance lengths.

6.2 Design

The primary lining for the main bores and cross passages was permanent 32 MPa steel Fibre Reinforced Shotcrete and was designed to resist all ground loading. The approach to the design of the primary lining is summarised as follows:

- Primary lining was designed structurally as plain concrete
- No flexural capacity of steel fibres was used in the design

- The selected shape of the primary lining results in no tension in the lining (bending moments are resisted by axial forces in the lining)
- Areas of the lining with very low axial loads utilise the allowable tensile capacity of the concrete in accordance with Eurocode 2 to resist bending moments.

Fibres are included in the sprayed concrete due to the selected excavation and ground support technique. In particular fibre reinforcement allows the primary lining to be installed safely whereas the installation of steel mesh would require personnel to stand and operate under unsupported ground as well as introducing implications relating to manual handling regulations. The fibre reinforcement was used for improved constructability of the SCL lining by improving the initial behaviour of the shotcrete and preventing slumping or tearing of the material as it was placed ensuring a homogeneous lining was achieved.

Whilst the flexural capacity of the steel fibres was not used in the design, RILEM TC 162-TDF: 'Test and design methods for steel fibre reinforced concrete' was required for design of the main bore primary lining sections, to provide suitable partial factors applied to the compressive stress block of fibre reinforced sprayed concrete. The proposed partial factor outlined below is the same as for reinforced concrete to reflect the increased ductility provided by fibre reinforced sprayed concrete when compared to plain concrete. The design compressive strength of the fibre reinforced concrete (f_{cd-sfr}) was derived in accordance with the equation in Clause 3.1.6 (1) of Eurocode 2 as below:

$$f_{cd-sfr} = \alpha_{cc} f_{ck-sfr} / \gamma_c \quad (5)$$

where α_{cc} = coefficient taking account of long term effects on the compressive strength and of unfavourable effects resulting from the way the load is applied (a value of 0.85 was used in accordance with the UK National Annex); f_{ck-sfr} = characteristic compressive cylinder strength at 28 days; γ_c = partial safety factor for concrete, taken from Table 2.1 (1.5 for Persistent and Transient loading in accordance with the UK National Annex).

For all permanent loads, the primary lining will experience no tension. With the section fully under hoop compression the tensile capacity of the fibre reinforced sprayed concrete is not used in the design. The lining design was undertaken at the Ultimate Limit State (ULS) whereby the design value of the load in the lining had a load factor of 1.35 applied, as given in BS EN 1990 Eurocode—Basis of Structural Design.

7 DESIGN INNOVATION—PERMANENT PRIMARY LINING DESIGN

The principal innovation with the support measures is the design of the primary lining as permanent. This was possible due to a number of advances in tunnelling technology in recent years. Firstly, 'alkali free' set accelerators are now available with no loss in shotcrete strength with time. A recent innovation is the use of 3-D scanning survey equipment that provides excellent shape control for both excavation and spraying, and allows shotcrete lined tunnels to be constructed without lattice girders. This technique has been recently used successfully for the Heathrow T5 project (Williams et al, 2004).

Historically the inclusion of lattice girders meant the primary lining had to be considered temporary due to the corrosion potential of the steel lattice girder within the primary lining. Spiling is envisioned in several locations due to adverse soil layers. This will be carried out with self-drilling Glass Reinforced Plastic (GRP) dowels, again with no adverse durability issues. Some of the key aspects of a permanent primary lining relate to construction techniques and workmanship, such as the shotcrete mix design and accelerator selection and use of robotic spraying equipment.

The ECI project delivery process allowed the designer to work hand in hand with the contractor during the design development phase, thereby ensuring all the necessary issues were addressed.

8 SHOTCRETE MIX SPECIFICATION

In order to achieve a durable shotcrete mix suitable for use as a permanent lining the specification included requirements for the base mix concrete, shown in Table 1, as well as specific fibre reinforced shotcrete requirements, included

Table 1. Base mix design requirements.

Requirement	Unit	Specified value
Intended working life of structure	yrs	120
Compressive strength (f'_c)	MPa	32
Minimum cementitious content	Kg/m ³	360
Maximum w/c ratio	—	0.45
Maximum aggregate size	mm	14
Chloride content class	—	Cl, 0.3
Maximum cementitious content	Kg/m ³	450
Maximum temp. of fresh concrete	°C	20
Water penetration to BS EN12390-8	mm	50

in Table 2. The other critical element included in the specification was the requirement for robotic spraying of the shotcrete. The durability of the base mix was assured through the specification of a maximum w/c ratio of 0.45 along with a water penetration requirement of less than 50 mm.

The performance specification of the shotcrete was not particularly onerous as the design approach did not utilise the flexural toughness of the steel fibre reinforced shotcrete.

The tests specified for pre-production mix confirmation as well as production control are included in Table 3. The aggregate grading requirements are shown in Figure 7.

Table 2. Fibre reinforced shotcrete requirements.

Structural performance requirement	Time	Specified value
Compressive strength	1 hr	Upper J2
	8–16 hrs	Upper J2
	24 hrs	10 MPa
	7 days	27 MPa
	28 days	32 MPa
Energy absorbtion (Plate test to EN14488-5)	28 days	700 J
Drying shrinkage to ASTM C157	28 days	<0.03%
Maximum w/c ratio	–	0.45
Maximum aggregate size	mm	14
Chloride content class	–	Cl, 0.3
Maximum cementitious content	Kg/m ³	450
Maximum temp. of fresh concrete	°C	20

Table 3. Shotcrete testing requirements.

Requirement	Pre-production testing	Production testing frequency
Compressive strength	Yes	Every advance (1, 8, and 24 hr) 1 set/100 m ³ (28 and 90 days)
Water penetration	Yes	1 set/month
Homegenity	Yes	1 test/week
Energy absorption	Yes	1 set/month
Flexural strength & toughness	Yes	–
Shrinkage	Yes	–
Density		1 set/100 m ³
Thickness		5 /shift

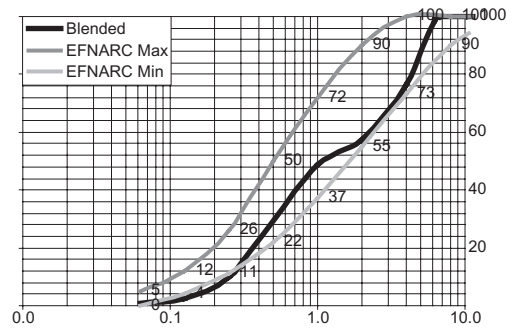


Figure 7. Aggregate grading curve.

9 CONSTRUCTION

The sprayed concrete lining was sprayed robotically using a BASF Meyco Potenza chassis with Logica laser scanning systems. The maximum throughput was 25 m³ per hour.

The sprayed profile was very accurate. An as-built full laser survey of the tunnels was completed and the shotcrete profile was generally within ± 50 mm of design. The profile control was carried out using reflectorless measurement of the shotcrete when sprayed. The profile was excavated 100 mm bigger than design and the target was to spray 50 mm bigger than design. This was largely achieved although there was some overbreak in areas of instability. The following sections describe the details of the construction.

9.1 Shotcrete mix trials

Trials were carried out onsite over the winter of 2007/08 to verify that the sprayed concrete met the specification requirements prior to being placed in the permanent works. These trials consisted of spraying panels (both vertical and overhead) to observe strength gain, 28 day strength and workability. Concurrently with these tests, samples were sent away for permeability and flexural strength testing.

The conditions during the testing were onerous as low ambient temperatures were experienced. This lead to some challenges in obtaining the required early age strength. An oil-burning water heater was used to raise the water temp at the batcher to around 50°C. This resulted in a batched shotcrete temperature of around 20°C. This was instrumental in achieving the J2 early age strength curve and this temperature was achieved, even when batching with ambient temperatures below zero. Once inside the tunnel, the consistently higher ambient temperatures helped to achieve the required strengths at early age. There were no problems in achieving

the required long term strength. There were no problems in attaining the permeability requirements (tested using the depth of penetration under water pressure method). Energy absorption testing was to EN14488-5 (former EFNARC panel test). Panels were sent away for testing and the optimum fibre dosage selected based on the results. The shotcrete mix design is shown in Table 4 below, with the aggregate grading curve included in Figure 7.

The shotcrete mix initially included 40 kg/m³ steel fibres but was reduced to 30 kg/m³. Alternative steel fibres at 35 kg/m³ were also approved. Additional trials proved that macro-synthetic polypropylene fibres at 7 kg/m³ met the performance specification requirements and subsequent testing allowed a reduction to 6 kg/m³. Alternative macro-synthetic polypropylene fibres were also approved at 6 kg. The macro-synthetic polypropylene fibres were 55 mm long and were sourced from various suppliers (Bekaert, Barchip and Propex). The shotcrete mixes were easier to apply with steel fibres but the supply proved unreliable and more expensive than polypropylene.

9.2 Progress rates

Tunnelling was undertaken concurrently in each bore from the northern portal on a 24-hour basis. Tunnelling was undertaken from the southern portal on day shift only, with crews alternating between bores for the canopy tube installation, and the excavation and spraying. Tunnelling commenced in February 2008, and both headings broke through on 26 February 2009. Peak combined progress on all faces exceeded 100 m per week. Table 5 includes the sprayed concrete lining production rates achieved.

9.3 Survey control

Excavation profile during excavation was maintained by reflectorless measurement from total stations. Target arrays were installed in the primary

Table 5. Sprayed concrete production rates.

Production rates	South (m)	North (m)	Comb. total (m)
Average heading/day	1.3	4.3	14
Best day	4.0	16.2	20
Best week	20.1	95.8	115
Best month (Heading)			457
Best month (Bench)			1232

lining at 25 m centres and surveyed back to control stations outside the tunnel portal. These target arrays were used to position the total stations. Primary lining profile control was also carried out using reflectorless measurement from total stations. The initial layers were placed using the previous lining as a guide, with the total station being brought in for guidance of the final passes. It is worth noting that whilst the reflectorless measurement technique allows points to be measured close to the face, it is susceptible to dust and overspray (causing apparent crown level drops) and blocking by plant/equipment.

Balfour Beatty has been working with suppliers to investigate the use of laser-scanning guidance systems which can be linked to the both the excavator and sprayed concrete robot. These would allow control of both the excavated and sprayed profiles without the need for personnel monitoring the profile in close proximity to the face. Currently, it is not clear that the systems provide additional accuracy although there are safety benefits associated with removing personnel from the tunnel.

The use of laser scanning for control of the internal profile of the sprayed concrete primary lining was not successful, and was profile was achieved by the separate survey system described above. Where a constant thickness is required (eg. secondary lining), the laser scanning system appears more capable although variations in concrete flow can cause variations in thickness due to inherent assumptions made by the machine about concrete flow rates.

9.4 Construction issues

The key advantage of the sprayed concrete support design is the degree of inherent flexibility it affords. This flexibility was increased by the development of certain structural items not originally envisaged in the design. These include temporary cross passages, the ability to excavate almost all of the northbound tunnel heading-only, turning bays at full depth constructed using the bench profile

Table 4. Shotcrete mix design.

Component	Quantity/m ³ or product
OPC cement	450 kg
Microsilica	40 L
Coarse aggregate 2/6	413 kg
Fine aggregate	1240 kg
Water	0.38 w/c
Superplasticiser	Glenium 51
Stabiliser	Delcocrete
Accelerator	Meyco SA 160
Fibres	30 kg steel fibres or 6 kg structural fibres

and a full-closed heading developed for the sand encountered at the south end. Each of these designs were produced by adapting the flexible base design and so were developed using minimal analysis whilst maintaining a safe and optimised design. The design elements were able to be installed as required by the contractor, allowing flexibility of programme onsite.

The design was optimised with the level of ground support provided being adjusted depending on ground conditions encountered at each face. Whilst this requires careful monitoring and control, the benefits were great (tunnelling with optimised support from the north portal was around four times faster than that from the south which used full support). As the site team developed a greater knowledge of the ground and tunnel behaviour, increased optimisation of the support level was possible. This included the use and type of spiles (32 mm solid GRP spiles installed into pre-drilled holes were the preferred option), the type of face support (an inclined face was the preferred option) and the installation of an invert strut.

10 CONCLUSIONS

In the 10 years since the previous bored highway tunnel was designed and constructed in the UK, there have been several advances in tunnelling technology. The A3 Hindhead project has adopted several new approaches and details, in order to minimise both the project risk and cost including the first use of fibre reinforced shotcrete as a permanent lining in the United Kingdom. This has been possible due to the:

- Availability of 'alkali free' set accelerators,
- Use of 3-D laser scanning survey technology for control of robotic spraying equipment to achieve tight shape tolerances avoiding the need to use lattice girders,
- Use of self-drilling GRP spiles in areas of poor ground.

The analysis and design of the lining included 2-D analysis as the primary design tool, as well as using 3-D analysis for the time-related effects. Design was undertaken for the Ultimate Limit State within the Eurocode 2 framework, although a revision to the material reduction factors was required to reflect the ductile nature of fibre reinforced shotcrete. Although the durability of the lining structure is greatly enhanced by avoiding the

inclusion of steel reinforcement, a durable shotcrete mix was assured through the specification of a low permeability concrete with a low water: cement ratio.

Construction proved to be very successful, with consistent high quality shotcrete provided at a rapid rate by the shotcrete robots. A combined production rate of over 100 m per week indicates that the method is very economical where multiple headings can be excavated concurrently.

Whilst sprayed concrete linings have been used typically in soft ground, this project has proved that this approach can be very economical for soft rock conditions. This lining was installed using NATM principles such that the majority of the load was taken by the ground, resulting in a relatively thin 200 mm thick lining. Where difficulties exist in developing tensile loads in bolted support systems due to low bond stress values, or in highly jointed rock masses, then SCL linings provide an economical but safe tunnel design alternative. These economics improve significantly when the primary lining is designed as a permanent lining. These conclusions are directly applicable to the soft bedrock material that exists in Auckland, the East Coast Bays Formation (ECBF).

REFERENCES

- Bieniawski, Z.T. 1984. *Rock Mechanics Design in Mining and Tunnelling*. Rotterdam: A.A.Balkema.
- Chang & Stille. 1993. Influence of early-age properties of shotcrete on tunnel construction sequences. *Shotcrete for Underground Support VI—Proceedings of Engineering Foundation Conference*.
- John, M. & Mattle, B. 2003. Shotcrete Lining Design: Factors of Influence. *Rapid Excavation and Tunnelling Conference*, pp. 726–734.
- Marinos, P.G. & Hoek, E. 2000. GSI: A geologically friendly tool for rock mass strength estimation. *Proceedings of the International Conference on Geotechnical and Geological Engineering (GeoEng 2000)*, Technomic Publishing Co. Inc., pp. 1422–1440, Melbourne, Australia.
- Osterreichischer Betonverein. 1999. *Sprayed Concrete Guideline—Application and Testing*. Nelkengasse. Ing. Rudolf Bankl.
- Williams, I., Neumann, C., Jäger, J. & Falkner, L. 2004. Innovativer Spritzbeton-Tunnelbau für den neuen Flughafenterminal T5 in London. In *Proc. Österreichischer Tunneltag 2004*, Austrian Committee of the ITA, Salzburg, pp. 41–62. Salzburg: Die SIGN Factory.

Shotcrete with blended cement and calcium aluminate based powder accelerator for improved durability

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ABSTRACT: The use of Fly Ash (FA) and Blast Furnace Slag (BFS), together with a powder accelerator based on calcium aluminate, were investigated to determine their effect on the durability of shotcrete. Blended cements of ordinary Portland cement and FA and BFS decrease the generation of Calcium Hydroxide (CH), which easily reacts with aggressive chemicals in aqueous environments such as those associated with marine structures and undersea constructions. The reaction of CH in a concrete structure leads to erosion and thus lowers the durability. The calcium-aluminate-based powder accelerator is hydro-reactive and promotes quick setting and strength development for blended cements with a low portion of ordinary Portland cement. In an experiment, the amount of CH in shotcrete containing FA and BFS decreased to one half that in shotcrete containing ordinary Portland cement. The compressive strength of the shotcrete exceeded 50 N/mm² after 28 days. Shotcrete containing FA and BFS, together with calcium-aluminate-based powder accelerator, has low CH generation and a high degree of strength development, which results in high resistance to seawater leading to a shotcrete that is suitable for application in construction exposed to sea water or submerged in the sea.

1 INTRODUCTION

Concrete is required to be highly durable especially when it is used in severe conditions such as in marine and undersea constructions. High-strength concrete with a dense microstructure and low concentration of calcium hydroxide (CH) is generally proposed for improving durability against seawater. The blending of cements with fly ash (FA) and blast furnace slag (BFS) decreases CH generation and is one of the solutions for improving the durability of concrete.

Although the use of blended cements has been tested and designed in the laboratory and applied in tunneling projects (Yoshida et al. 2006; Yamamoto et al. 2002; Haga et al. 2002), the application of shotcrete using blended cements in tunneling projects has been limited compared with the case for concrete in other construction fields. The major reason for the limited use of blended cements in shotcrete is that they detrimentally affect the important properties of quick setting and early strength development. Powder accelerators in which the main component is calcium aluminate are hydro-reactive and promote quick setting and early strength development in shotcrete. The quick-setting property is highly advantageous for blended cement shotcrete containing less hydro-reactive Portland cement. In this paper, the quick-setting and strength development properties of shotcrete comprising

blended cement and a powder accelerator are evaluated together with the calcium hydroxide generation.

2 MATERIAL DESIGN AND COMPOSITION OF SHOTCRETE CONTAINING BLENDED CEMENT

2.1 Concrete additive and blended cement

BFS, FA and silica fume are the main materials used as additives or Supplementary Cementitious Materials (SCM) in concrete applications and are widely used for blended cements. Blended cements containing BFS, fly ash, and pozzolan (silica fume) are standardized under Japanese Industrial Standards (JIS) R 5211-2003, R 5213-1997, and R 5212-1997.

Among these materials, BFS is the most widely used additive for concrete application in Japan, and it is standardized under JIS A 6206-1997. Three types of BFS with different Blaine specific surface areas, ranging from 3000 to 10,000 cm²/g, are standardized for concrete application. For blended cement, BFS with a Blaine specific surface area from 3000 to 5000 cm²/g is used.

Fly ash is standardized under JIS A 6201-1999, and four types of fly ash with different Blaine specific surface areas ranging from 1500 to over 5000 cm²/g are standardized for concrete application. Type I with a Blaine specific surface

area exceeding 5000 cm²/g is used for blended cement.

2.2 Accelerator for shotcrete

Shotcrete has been applied in many constructions and for underground support with various types of accelerators. Liquid accelerators can be classified into two groups: conventional alkaline accelerators and the newly invented acid accelerators—the so-called alkali-free liquid accelerators. Several types of powder accelerators are also known: alkaline accelerators in which the main component is an inorganic salt such as alkaline aluminate or carbonate, and cementitious mineral accelerators in which the main component is calcium aluminate.

Powder accelerators based on cementitious minerals promote the hydration of Portland cement, while the accelerator itself is hydro-reactive and hardened (Teramura et al. 1993; Hirose & Yamazaki 1993). Thus, compared with other types of accelerators such as inorganic-salt-based chemicals and aluminum-sulfate-based liquid accelerator, cementitious-mineral-based accelerators have superior setting and early strength development properties. Accordingly, they have advantages in applications within ground that has complicated geological formations and much ground water.

Materials combining calcium oxide, aluminum oxide, and sulfate are used as special cement additives to improve the performance of concrete. Quick setting, rapid hardening, high strength, and expansion or shrinkage compensation are achieved by controlling the period of ettringite (3CaO·Al₂O₃·3CaSO₄·32H₂O) formation during cement hydration (Ishida, 2000). For example, the quick setting of cement or concrete is achieved by the formation of ettringite in the very first stage of cement hydration.

From the above material design considerations, to improve the performance of concrete an accelerator to increase the strength development of shotcrete has been developed containing calcium aluminate and sulfate components. The accelerator generates a large amount of ettringite at an early stage, which enhances the quick-setting performance and early-age strength development (Ishida et al. 2001). For shotcrete containing blended cement, an accelerator that promotes high-strength development is preferred.

2.3 Mix design of a shotcrete with blended cement and powder accelerator

Table 1 shows the materials used in the evaluation and Table 2 shows the mix proportions of the

Table 1. Materials used in the evaluation.

Material (symbol)	Specification	Density (g/cm ³)
Ordinary Portland cement (C)	Blaine specific surface area: 3200 cm ² /g	3.15
Blast furnace slag (BFS)	Blaine specific surface area: 5000 cm ² /g	2.98
Fly ash (FA)	Blaine specific surface area: 5500 cm ² /g	2.2
Sand (S)	River sand, crushed, F.M. 2.90	2.62
Gravel (G)	<13 mm	2.64
Superplasticizer (SP)	Polycarboxylic-acid-type	1.04
Accelerator (Acc)	Calcium aluminate base with sulfate for high strength	2.8

Table 2. Mix proportion of shotcrete.

Type	W/B (%)	Water	Unit volume (kg/m ³)				
			Binder (C)+ (BFS) (FA)	S	G	SP	Acc.
B	40	200	500 (220) (180) (100)	946	709	4.8	32
H	45	203	450 (450) (0) (0)	1115	605	5.4	45

evaluated shotcrete. A powder accelerator based on calcium aluminate with a sulfate component is used to achieve high-strength development and high durability. A superplasticizer of polycarboxylic acid type was used to reduce the water to cement ratio for better strength development.

Blended cement containing BFS and FA was selected for the shotcrete evaluation, which is type B in Table 2. For comparison, shotcrete without BFS or FA was also evaluated, which is type H in Table 2.

3 EVALUATION OF FRESH CONCRETE AND SHOTCRETE

3.1 Fresh concrete evaluation: slump and workability

Table 3 shows the slump and slump flow of the fresh concrete. A high initial slump of 24 cm was

Table 3. Slump and slump flow of the fresh concrete.

Type	Slump (cm)			Slump flow (cm)		
	0 min	30 min	60 min	0 min	30 min	60 Min
B	24.0	24.0	22.5	47.5	49.0	42.0
H	18.5	—	—	—	—	—

Table 4. Quick-setting property of mortar with accelerator, Proctor penetration resistance (MPa).

Type	45 sec	1 min	2 min	3 min	5 min	7 min
B	5.7	9.0	17	20	24	28
H	17	25	36	—	—	—

selected for this evaluation. Suitable workability was maintained for 60 minutes.

3.2 Mortar evaluation: quick setting property

The quick setting property was evaluated using mortar having mixture proportions such that gravel was eliminated from the concrete mixture. Powder accelerator was added to the fresh mortar and mixed for 15 seconds in a mixer, and then the mixture was immediately filled into a mold. The Proctor penetration resistance in accordance with ASTM C-403 was applied for the evaluation. The times of initial set (resistance of 3.5 MPa) and final set (resistance of 28 MPa) were found. The measurement was carried out at 20°C.

Table 4 presents the results of the testing of the quick-setting property. Both mortars set quickly, achieving an initial set before 45 seconds and a final set before 10 minutes, compared with 7 minutes for type B and 2 minutes for type H. The quick-setting property is closely related to shotcrete properties of adhesion to the ground and the thickness that can be sprayed at one time. Both setting times indicate suitable properties of the mixture. With such a quick-setting property, usually more than a 20 cm thickness can be sprayed on overhead ground on one pass.

3.3 Shotcrete evaluation: strength development

Table 5 shows the test items and method for shotcrete evaluation. For evaluation of early-age strength development after 3 hours and one day, the pull-out method in accordance with JSCE-G562 (2005) was used. The compressive strength of shotcrete after 7 days was measured by taking a sample core from a shotcrete panel applied to a frame with dimensions of 50 × 50 cm. The test

Table 5. Test items and method for shotcrete evaluation.

Item	Parameter	Method	Age
Fresh concrete	Slump	JIS A 1101	
	Air content Concrete temperature	JIS A 1128 Thermometer	
Base concrete	Compressive strength Ø10 × 20 cm	JIS A 1132, 1108 Water curing	7 days, 28 days, 91 days
	Compressive strength by pull out	JSCE G 562 Curing at 20°C	3 hours, 1 day
Shotcrete	Compressive strength by core	JIS A 1107 Water curing	7 days, 28 days, 91 days

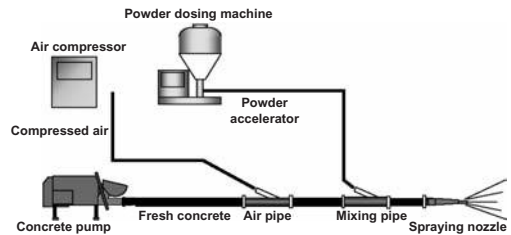


Figure 1. Schematic diagram of the shotcrete system for the wet process using powder accelerator.

piece was 55 mm in diameter and 110 mm long, and the core strengths at 7 and 28 days were measured in accordance with JIS A 1107 (1993).

The compressive strength of the base concrete was measured for a test sample 100 mm in diameter and 200 mm long. The test sample was prepared by filling the fresh concrete into a mold of the test sample size and then removing from the mold after one day and curing in water at 20°C.

3.4 Equipment and system for the evaluation of shotcrete

Shotcrete was evaluated in a model tunnel 4.5 m high, 5.5 m wide and 20 m long. The wet process was applied, and a piston-type concrete pump with a maximum capacity of 25 m³/h was used in the experiments.

Figure 1 is a schematic diagram of the shotcrete system for the wet process using powder accelerator. Powder accelerator is added to wet-mix shotcrete using an air conveyance system and a supply device similar to that used in rotary-system equipment. The accelerator is conveyed with pressurized air and mixed with wet concrete at the

Table 6. Strength development of shotcrete (MPa).

	Pullout		Core			Base concrete		
	3h	1 d	7 d	28 d	91 d	7 d	28 d	91 d
B	2.8	10.3	30.2	44.5	56.2	31.4	49.6	66.4
H	2.5	17.8	38.0	48.2	–	–	49.9	–

Y-intersection within the mixing pipe, before being sprayed from the nozzle.

3.5 Results of shotcrete evaluation

Table 6 shows the strength development of the shotcrete. The shotcrete containing blended cement exhibited a strength of about 3 MPa after 3 hours, 10 MPa after 1 day, and over 40 MPa after 28 days. The strengths were nearly the same as those for type H after 3 hours and 28 days, but differed from those for type H after 1 day and 7 days. As the blended cement contains less Portland cement, hydration at these ages is presumed to be less than that for shotcrete containing ordinary Portland cement (OPC), resulting in lower strength development.

4 CALCIUM HYDROXIDE IN SHOTCRETE

4.1 Evaluation of calcium hydroxide content in cement paste and shotcrete

The CH content was evaluated for three types of samples: cement paste without accelerator, shotcrete with accelerator, and concrete without accelerator. Cement paste was prepared without sand and aggregates. Sixteen samples were prepared with different OPC contents, BFS contents and FA contents. Shotcrete samples were prepared from test specimens for analysis of the core strength, and a concrete sample without accelerator was prepared from test specimens for compressive strength analysis.

Each sample was crushed into small grains of a few millimetres and hydration was stopped after 28 and/or 91 days using acetone. Samples were ground to powder and the CH content was measured by thermogravimetry differential thermal analysis. Table 7 shows the CH content of cement paste after 28 days. The CH decreased as the OPC content decreased and the FA content increased. The CH content decreased to about half the normal content for an FA content of 300 kg/m³ in the blended cement. There was no clear effect of BFS on a decrease in the CH content. The effect of BFS and FA are discussed later regarding cement hydration.

Table 8 and Figure 2 show the CH contents of the shotcrete and base concrete (shotcrete without

accelerator). The percentage measurements indicate the CH content in the cement paste of the concrete. The measurements with units of kg/m³ indicate the CH content per 1 m³ of concrete. Shotcretes and base concretes of type B have low CH contents compared with type H mixtures. In addition, the CH content of the shotcrete is less than that of the base concrete for both type B and H mixtures. The reason for the low CH content in shotcrete is not clearly understood. Hydration of the powder accelerator with Portland cement needs to be investigated to understand the low CH content in the shotcrete.

Table 7. Calcium hydroxide content of cement paste with different OPC, BFS and FA contents.

OPC (kg/m ³)	BFS (kg/m ³)	CH content (%)			
		FA 0 (kg/m ³)	FA 100	FA 200	FA 300
360	0	16.92	13.52	10.69	8.22
200	160	9.25	6.76	5.36	4.33
220	180	9.32	7.09	6.41	5.10
250	200	9.80	7.74	6.26	4.80

Table 8. Calcium hydroxide content of the shotcrete sample and concrete without accelerator.

Type	Concrete	CH content at 28 days		CH content at 91 days	
		%	kg/m ³	%	kg/m ³
B	Base	2.05	14.43	1.75	12.32
	Shotcrete	1.32	10.26	0.78	6.04
H	Base	6.65	38.32	5.92	34.11
	Shotcrete	3.86	23.22	4.04	24.31

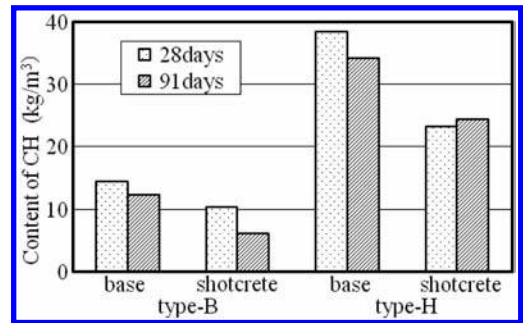


Figure 2. Calcium hydroxide content of the shotcrete sample and concrete without accelerator. The type-B mix contains cement and FA/BFS, while the type H mix contains only cement as a binder.

4.2 Cement chemistry and calcium hydroxide generation in concrete, with and without blast furnace slag or fly ash

The major components of OPC are the calcium silicates $3\text{CaO} \cdot \text{SiO}_2$ and $2\text{CaO} \cdot \text{SiO}_2$, both of which react with water to produce C-S-H ($n\text{CaO} \cdot \text{SiO}_2 \cdot m\text{H}_2\text{O}$, with a CaO/SiO_2 molar ratio of about 1.7) and CH. The amount of CH produced by 100% hydration of OPC is reported to be about 25% of the OPC content. In the case of the OPC proportion in a concrete mix being 220 kg/m^3 , as for shotcrete type B, about 55 kg/m^3 of CH is produced.

Table 9 shows the typical compositions of BFS and FA. As BFS and FA are potential hydraulic substances, both react with CH to produce C-S-H ($n\text{CaO} \cdot \text{SiO}_2 \cdot m\text{H}_2\text{O}$, with a CaO/SiO_2 molar ratio of about 1.5, which differs from that of the C-S-H in cement hydration). The amounts of CH that react with BFS and FA change with the SiO_2 and CaO contents. BFS contains about 40% CaO and reacts with SiO_2 during hydration. FA reacts with more CH than BFS does. From calculation, in the case of BFS, the amount of CH which reacts with BFS and produces C-S-H is about 10% of the BFS. In the case of FA, that of CH is about 100% of FA. To reduce the generation of CH, FA is much more effective than BFS.

Table 10 shows the balance of CH for the two shotcrete mix proportions evaluated in this study. From calculation, no CH remains in shotcrete type B, which contains blended cement. The difference between the measured results (Table 8) and calculated results (Table 10) may arise

Table 9. Typical compositions of BSF and FA.

Item	Content (%)	
	SiO_2	CaO
Blast furnace slag	35	40
Fly ash	40–70	0–10

Table 10. CH balance with different OPC, BFS and FA contents (kg/m^3).

Cement			Consumed CH				
OPC (a)	BFS (b)	FA (c)	CH gener- ation (25% of (a))	By BFS 10% of (b)	By FA 100% of (c)	Total	Bal- ance
220	180	100	55	18	100	118	–63
450	0	0	113	0	0	0	113

from the hydration speed and the movement of material by solid–liquid phase reaction. Hydration of OPC and potential hydraulic substances continues for several years and CH is generated and consumed gradually. Detailed investigations of chemical aspects in this matter are expected in the future.

5 SHOTCRETE CONTAINING BLENDED CEMENT USED IN TUNNEL PROJECTS

Shotcrete containing blended cement with FA has been applied in many tunnel projects in Japan. In the Shin–Tamayama tunnel and Kamosaka tunnel projects, cement blended with 14 to 17% FA was applied with a powder accelerator based on calcium aluminate (Haga et al. 2002). In the Sakaigatani road tunnel project, cement blended with 27% FA was applied with a slurry-type accelerator based on calcium aluminate (Iwaki et al. 2007). In the Kusuda shinkansen railway tunnel project, cement blended with 20% FA was applied with a powder accelerator based on calcium aluminate and a slurry-type accelerator (Hirama et al. 2007).

Compared with the use of shotcrete containing FA, the application of shotcrete containing BFS is limited. Miyauchi et al. (2002) reported the characteristics of the high strength shotcrete tested at Kanaya highway tunnel project using cement blended with more than 40% BFS.

6 CONCLUSIONS

Fly ash and blast furnace slag, together with powder accelerator based on calcium aluminate and sulfate components, were applied to shotcrete to improve durability. In our experiment, the amount of calcium hydroxide in the shotcrete decreased to one half of that in shotcrete containing only OPC. The compressive strength of the shotcrete exceeded 50 N/mm^2 after 28 days. Shotcrete containing fly ash and blast furnace slag, together with calcium-aluminate-based powder accelerator, exhibits low calcium hydroxide generation and a high degree of strength development, which results in high resistance to seawater and means the shotcrete is suitable for application in construction exposed to sea water or submerged in the sea.

REFERENCES

- Haga, H., Yokoya, K., Abe, K. & Aoya, F. 2002. “Development of high-quality shotcrete using coal ash and application of tunneling projects”, *Proceedings of Shotcrete for Underground Support IX*, 271–280.

- Hirama, A., Iwaki, K., Yanagimori, Y., Noda, H. & Kariya, K. 2007. "High quality shotcrete containing fly ash in Kusuda tunnel (Kyushu Shinkansen)", *Tobishima Gihou*, vol. 56, 1–7.
- Hirose, S. & Yamazaki, Y. 1993. "Hydration properties of shotcrete with an accelerator based on calcium aluminate", *Proceedings of Shotcrete for Underground Support VI*, 25–32.
- Ishida, A. 2000. "Special cement additives using ettringite formation", *Transaction of the Materials Research Society of Japan*, 25[2], 561–564.
- Ishida, A., Teramura, S., Hirano, K. & Nakagawa, K. 2001. "Calcium sulfo-aluminate based accelerator for high-strength shotcrete in Japan", *Proceedings of Shotcrete for Underground Support VIII*, 202–215.
- Iwaki, K., Yasunaga, R., Ishimoto, T., Higashi, S. & Hirama, A. 2007. "Application of shotcrete using JIS grade I fly ash and slurry type accelerator in Sakai-gatani tunnel (Shimanto tunnel)", *Tobishima Gihou*, vol. 56, 8–17.
- JIS A 1107, 1993. "Method of obtaining and testing drilled cores and sawed beams of concrete", *Japanese Industrial Standards Committee*.
- JIS R 5201, 1997. "Physical testing method for cement".
- JSCE-G 561, 2005. "Test method for early strength of sprayed concrete (mortar) by pull-out method", *Japan Society of Civil Engineers*.
- Miyauchi, T., Itoh, K., Ohshima, H., Ohnaka, T., Matsuda, A., Yasui, Y. & Suda, H. 2002. "The characteristics of high strength shotcrete using the portland blast-furnace slag cement", *Proceedings of the 57th Annual Conference of the Japan Society of Civil Engineers*, 5, V-491, 981–982.
- Teramura, S., Matsunaga, Y., Hirano, K., Handa, M. & Sakai, E. 1993. "Accelerator for shotcrete based amorphous calcium aluminate", *Proceedings of Shotcrete for Underground Support VI*, 9–16.
- Yamamoto, K., Morioka, M., Teramura, S. & Sakai, E. 2002. "Design of mix proportion of shotcrete with low environmental burden (in Japanese)", *Cement Science and Concrete Technology*, No. 56, 148–155.
- Yoshida, S., Taguchi, F., Yamanaka, S. & Sato, H. 2006. "Application of shotcrete for NATM using blast furnace slag cement", *Proceedings of Shotcrete for Underground Support X*, 89–98.

Sprayable fire-protective layers in traffic tunnels

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ABSTRACT: Fires in tunnels are characterized by a rapid increase in temperature with maximum temperatures rising up to 1300 degrees Celsius. Thermal stresses, material deterioration and explosive spalling may reduce the load bearing capacity of a tunnel lining. For new tunnel constructions there are a number of measures that may be taken for coping with the load scenario resulting from such fire ingress. When it comes to rehabilitation of existing tunnels, the subsequent application of a protective layer is generally the only course available. Protective layers must safeguard the structure within defined time limits against excessive temperatures, but also withstand the rough tunnel environment and the alternating stresses of suction and pressure caused by passing traffic over long periods. Shotcrete with polypropylene micro-synthetic fibres (PP fibres) as well as sprayable lightweight mortars are successfully used as protective layers. Requirements, test results and recent applications in Austrian tunnels will be discussed.

1 INTRODUCTION

1.1 Tunnel fires

Fire disasters in traffic tunnels (Kusterle et al. 2004), such as the fires in the Storebælt Tunnel—1994, in the Euro Tunnel—1996, 2008, in the Mont Blanc Tunnel—1999, in the Tauern Tunnel—1999 and in the Gotthard Tunnel—2001, not only interrupted important European traffic connections for a long time but also severely damaged the concrete inner lining of these tunnels.

Goods transported through our tunnels by train and lorry may release energy at a rate of up to 300 MW when burning. These fires exceed temperature-time curves such as the ISO 834 standard fire. Most impressive is the fast temperature rise within a few minutes to more than 1000° Celsius (Figure 1).

Concrete does not burn, but ordinary concrete does not always handle fire very well. The high temperatures generated in hydrocarbon fuelled fires (gasoline or diesel fuel, cooking oils, animal fats, tyres, etc.) lead to a rapid temperature rise in close-to-surface layers, heat up pore-water and may quickly convert moisture in the concrete matrix into steam. Moisture in ordinary concrete is heated faster than it can migrate away from the heat source. As this process continues, the vapour pressure exceeds the concrete's tensile strength, and at this point, explosive spalling occurs (Tatnall, 2002).

Even if spalling does not reach deep parts of the structure, it will expose reinforcement directly to the effects of fire ingress at a very early stage, thus

reducing the load-bearing capacity of the structure (Figure 2). Besides spalling, temperature penetration will reduce the strength of concrete and reinforcement and lead to restrained stresses in the structure.

1.2 Influence of micro-synthetic PP fibres (ppf) on concrete exposed to fire

PP fibres cannot protect concrete from thermal deterioration. However, in case of fire ingress micro-synthetic PP fibres intermixed into the concrete can have the following effects.

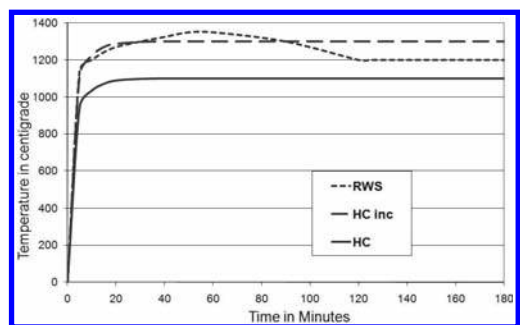


Figure 1. Temperature-time curves which have to be applied for testing and design according to OEVB (2005), OEVB (2006) in Austria (HC and HC increased for hydrocarbon fires with different duration, if detailed fire design is not performed, RWS (Rijkswaterstaat, a Dutch requirement) for proving fire resistance class BBG (fire resistant concrete tested in large scale test, "Beton erhöhter Brandbeständigkeit geprüft im Großversuch") and for protective layers).

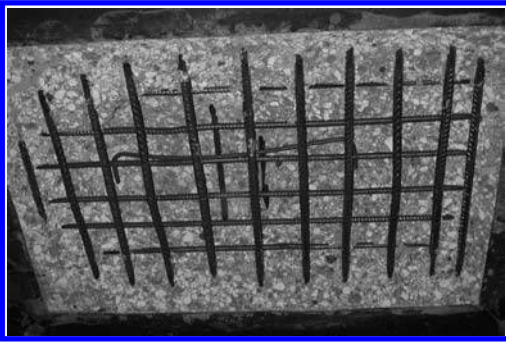


Figure 2. Reinforced concrete test panel after fire ingress showing heavy material loss due to spalling (Kusterle 2004).

PP fibres at normal dosage levels for FRC do not significantly change the thermal conductivity of concrete, but the addition of micro-synthetic fibres may lead to a cooling and pressure relief effect within moist concrete because of additional possibilities for the water vapour to escape (Kusterle et al. 2004; Waubke, 1973). It has been recognized that the temperature distribution within micro-synthetic fibre-reinforced high performance concrete depends on the micro-synthetic fibre content (Wille et al. 2003).

A sufficient volume of micro-synthetic PP fibres (and maybe other polymer fibres) can protect close-to-surface layers of concrete from explosive spalling (OEVB, 2005). Persson (2006) states “In order to estimate the required amount of PP fibres (ppf) more than 300 tests over 20 years were evaluated to estimate the effect of the diameter of fibres, dimensions of the structure, moisture level, heating rate, loading level, reinforcement, relative humidity, surface mesh reinforcement, w/c, and so forth. These numerous test results and recommendations from fire resistance tests prove the function of ppf”.

But the exact mechanism of the way micro-synthetic fibres act is still unknown, even if many theories exist (see Section 2). This is a major drawback: as the working mechanism is unknown, the creation of a tailor-made fibre for this purpose is not possible at the moment. Note that the addition of fibres may also influence other fresh and hardened concrete properties.

2 EXPLOSIVE SPALLING

2.1 Definition and reasons for explosive spalling

Generally, the detaching of concrete fragments as a consequence of exposure to fire is defined as spalling. Three different kinds of spalling can be identified (Kordina & Meyer-Ottens, 1981, 1999):

- Explosive spalling of close-to-surface concrete layers,
- Sloughing off,
- Aggregate spalling.

In the following, only explosive spalling will be addressed as ‘spalling’, as the other two types of spalling do not have any practical importance. “Rapid heating is necessary for spalling of traditional concrete. The rapid heating gives rise to large temperature and moisture gradients in the fire-exposed parts” (Hertz, 2003). Water and water vapour in the pore system are the main reason for explosive spalling. But this is a dynamic system (Wetzig, 2000a), since the formation of water vapour takes place as a function of time, and part of the water vapour can escape via the pore structure that is present. Damage only occurs when more water vapour is formed in the matrix than can escape via the pore structure.

Fire spalling is therefore influenced, among other parameters, by:

- the moisture content of the concrete
- the porosity of the concrete (w/c, strength class) and
- the temperature gradient (heating rate),

and also by

- the mechanical stress level,
- the fibre content (steel or plastic fibres) and the mix design as well as the constitutive materials used (aggregates, fines...),
- the geometry and dimensions of the structure,
- the reinforcement layout and concrete cover.

2.2 Common explanations

Most authors agree that water is the main cause of spalling: “All other reasons mentioned may contribute to the effect of spalling, but cannot cause spalling without moisture” (Hertz, 2003). The moisture in the pore system and the physically and chemically bound water in the concrete evaporate at high temperatures, leading to an increase of vapour pressure in the concrete structure. After evaporation of water the vapour is advected towards the fire-exposed surface as well as into the concrete structure where it cools down and condenses. As a result, a quasi-saturated layer is formed which is quasi-impermeable for water vapour (also known as a “moisture clog”, Kalifa et al. 2000, see Figure 3 from Schneider & Horvath, 2002). The magnitude of the stresses occurring essentially depends on the heating rate, on the amount of pore water (ratio of physically and chemically bound water) and on the pore structure through which the water vapour is transported. If the amount of water vapour

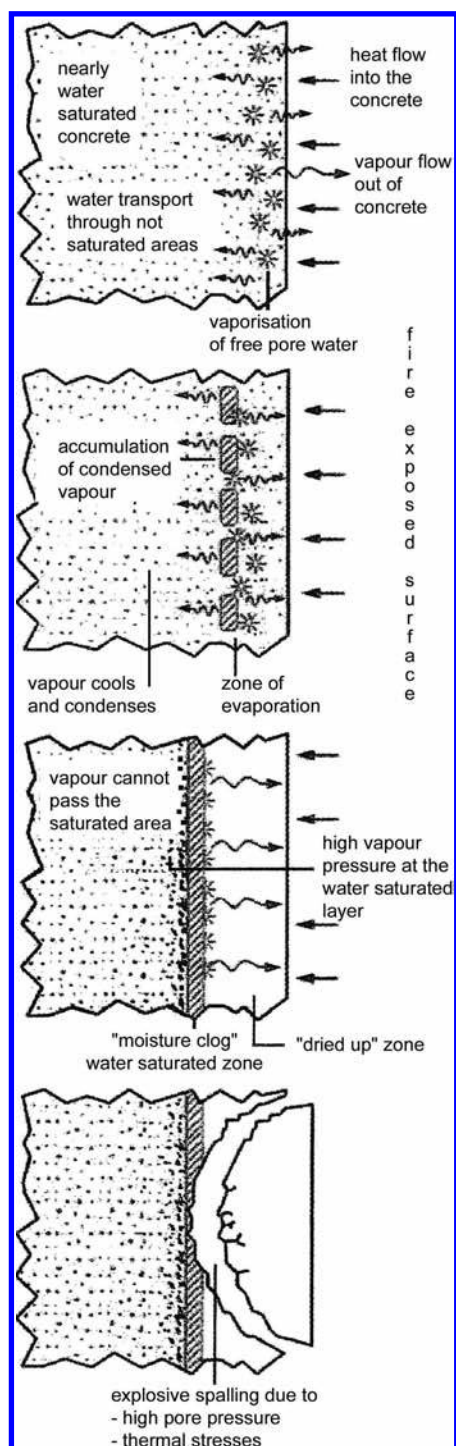


Figure 3. Schematic illustration of water vapour flow within a concrete structure heated from one side (Schneider & Horvath, 2002).

produced per unit of time exceeds the amount of vapour transported out of the pore structure, vapour pressure increases within the layer in which the evaporation occurs. With increasing pressure more water vapour escapes from the concrete which means that the vapour pressure and the amount of advected water vapour depend upon each other. The position and the time of evaporation depend on the “history of evaporation”. A condition of critical vapour arises at a temperature of 374°C. Beyond this point a pore space cannot contain liquid and vapour at the same time, and pressure increases dramatically (Hertz 2003).

Sources of water in concrete are the evaporable water (e.g. physically bound water), dehydration (of non-evaporable water, e.g. calcium hydroxide and C-S-H), and dissociation (e.g. of calcium carbonate) taking place during the heating process. Accordingly, normal-strength concrete cannot spall at moisture contents lower than 2% or 3% by mass and below a certain moisture content no other possible causes can lead to spalling. Kordina & Meyer-Ottens (1981–1999) indicate a moisture content <2%. This value seems more reliable in the light of the results of a recent research project (Kusterle et al. 2004).

There are alternative theories explaining possible causes for spalling as a result of evaporation of water. On the one hand, there is the assumption that a static pressure within the pores will lead to spalling if the tensile strength of concrete is exceeded (Meyer-Ottens, 1972). According to Schneider et al. (2001) the tensile strength of normal strength concrete may be reached due to the increasing vapour pressure at temperatures of approximately 250°C. On the other hand, the fluid transport within the concrete is considered to cause tensile stresses leading to spalling. It is also explained (Florian, 2002) that the expansion of water during heating causes a pore pressure increase just before evaporation occurs. However, it is not stated whether this might cause explosive spalling.

According to Kalifa et al., “spalling results from two concomitant processes: the so-called thermo-mechanical process, associated with the thermal dilation/shrinkage gradients that occur within the element when heated, and the thermo-hydral process that generates high-pressure fields of gas (water vapour and enclosed air) in the porous network” (Kalifa et al. 2001).

2.3 Other influencing factors on spalling reported in literature

One additional factor that may encourage spalling is internal stress caused by the heterogeneity of concrete. The different thermal expansion coefficients α_T of a steel reinforcing bar and the concrete

may lead to dissolution of bond and to cracks occurring around the reinforcing bar which consequently may favour spalling (Meyer-Ottens, 1972). According to Schneider & Horvath (2002) internal stresses may also occur within the concrete (hardened cement paste and aggregates). Internal stresses may also be caused by restrained dilation due to the specimen geometry. As a result of energy intrusion into the structure and the resulting heat penetration curve the expansion rate within the concrete structure varies with depth. Considering the condition of deformation compatibility this will lead to compressive stresses in the heated zones and to tensile stresses in the orthogonal direction to the direction of heat penetration. However, it has been claimed (Meyer-Ottens, 1972) that exceedance of compressive strength on the fire-exposed surface certainly does not lead to spalling. It is suggested (Ulm et al. 1999) that internal stresses, as described above, may promote spalling.

Restrained stresses can occur if dilation (e.g. due to great distances between movement joints) or free rotation of the structure (e.g. at the edges) is restrained, whereupon compressive stresses and tensile stresses may occur which consequently could favour spalling. The general stress condition on the structure can also play a role. High compressive stresses reduce the number of cracks occurring within the concrete structure and therefore reduce the possibility of the vapour escaping. As a consequence the spalling rate will increase.

Chemical processes within the concrete can lead to damage of concrete as has been frequently described in the literature, (e.g. Schneider & Horvath, 2002; Ulm et al. 1999; Wetzig, 2000b; Kalifa et al. 2001). Depending on the temperature of the concrete, minerals of the hardened cement paste or the aggregates are chemically transformed. A theoretical analysis regarding the possible causes for spalling is presented by Paliga (2003).

2.4 Possible measures for preventing spalling

Concrete has a very good fire resistance. The thermal conductivity of concrete structures is low, which means that a concrete member is only heated in the close-to-surface layers while the inner layers show only small temperature increases. In order to achieve further improvements, especially regarding the spalling behaviour, various measures have to be taken. In consequence of test results (Kusterle et al. 2004) several different ways of avoiding explosive spalling of close-to-surface concrete layers may be identified.

2.4.1 Reduction of the temperature gradient

A barrier (fire protective layer) is able to reduce the temperature penetration into the structure and to

avoid explosive spalling by separating the structure from the fire. This barrier can be: (I) a plate-like protective layer (e.g. construction boards) mounted on the concrete surface or on underlying structures, or (II) fire protection mortar (sprayable lightweight mortar or micro-synthetic PP-fibre-reinforced shotcrete (Kusterle et al. 2006)) bonded to the concrete (Figure 4, for other materials see Section 3).

If fire protective layers are moist, they may also spall, an aspect not examined in many cases. Moreover, the performance of interface, which is influenced by compression and suction effects caused by traffic and by the dead weight of the protective layer, has to be considered. In the case of protective layers made of mortars, the fire load imposes stresses in the interface. Therefore, mesh reinforcements should be built in and anchored in the concrete. Moreover, long-term aspects of performance (e.g. moisture and frost resistance) and the difficulty of access to the construction for examination purposes should be considered. Fire protective layers will be addressed in detail in Section 3.

2.4.2 Reduction of concrete moisture

Assuming that the volume expansion of both liquid water and vapour is the main cause of explosive spalling, a reduction of the moisture content theoretically represents an effective protective measure.

2.4.3 Providing expansion space

In the case of great volume expansions of constituents of the concrete matrix, the existing stresses can be relieved by the escape of moisture through macroscopic cracks or microscopic pores (longitudinal pores serve as escape ways, spherical voids serve as expansion spaces).

The water vapour should preferably escape through the pores. For reasons of durability, the capillary porosity of concrete has to be kept as low as possible. Therefore, as far as normal-strength concrete is concerned, the measure described above

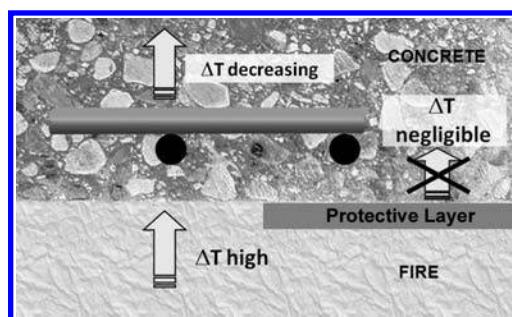


Figure 4. Reduction of the temperature gradient induced by fire to subcritical values by means of a protective layer.

is suitable for diminishing the existing vapour pressure only to a certain degree. Artificially entrained air bubbles provide a limited expansion space, which has been shown to be effective in case of volume expansion occurring as a consequence of frost. The usual air-pore content probably does not provide sufficient expansion space for vapour. Therefore, longitudinal pores are more suitable to serve as canals. However, they should serve as transport canals only in case of fire and prevent flow of air, water, and harmful substances under normal conditions. Research (e.g. Persson, 2006) has shown that micro-synthetic polymeric fibres (especially polypropylene fibres) possess these characteristics, even if the mechanism has not been clarified in detail up to now. They can be used in protective layers too.

2.4.4 Increase of the concrete tensile strength

Spalling could be delayed if the tensile strength of concrete is increased without increasing the compressive strength (and normally reducing porosity). But this is only possible to a limited extent, e.g. by the addition of steel fibres.

2.4.5 Reinforcement and reinforcement layout

Thin mesh reinforcement protecting the main reinforcement has only a minimal influence on spalling behaviour. However, reinforcement bars can serve as “support” for forming arches (supporting vaults) and as “barriers” for concrete parts broken off, if appropriate diameters and axis distances are chosen and the bars are anchored in deeper parts of the structure (Kusterle et al. 2004).

2.4.6 Providing space for dilation

Expansion joints may help to reduce spalling caused by restrained loads due to the restrained longitudinal expansion of structures.

3 PROTECTIVE LAYERS

Reducing the temperature gradient in the outer concrete layers (Section 2.3.1) is a very effective measure to protect concrete from spalling (Kusterle et al. 2006). According to EN 1992-1-2 (2007) protective layers are defined as “any material or combination of materials applied to a structural member for the purpose of increasing its fire resistance”. Recently the following products have become available:

- plate-like protective layers (construction boards) mounted on the concrete surface or on underlying structures,
- fire protective lightweight mortar bonded to the concrete,

- PP-fibre-reinforced shotcrete bonded to the concrete (OEVB Guideline for Shotcrete 2004),
- (coated, perforated steel sheets or enamelled steel sheets),
- (intumescent (self raising foaming) coatings).

The last two materials have not acquired any importance in tunnels due to operating requirements. Protective layers have to protect the structure from any detrimental temperature ingress for a specified time. But for the whole service life they have to withstand the rough tunnel environment and stresses from compression and suction effects caused by passing traffic. The operator of the tunnel requires access to the construction, a bright surface appearance and easy cleanability. For existing tunnels protective layers are the only way of upgrading the structure with regard to fire resistance.

3.1 Catalogue of requirements

Requirements on tunnel linings are different from those usually made for the application of protective layers in houses or industrial buildings. Therefore the same materials are rarely used for both purposes. The requirements may be subdivided into:

- fire resistance and insulation effects,
- bonding to the substrate, especially during suction,
- durability (especially due to water and frost action),
- accessibility to the main structure and visibility of new cracks in the structure,
- appearance and cleanability (mainly in road tunnels),
- costs of application and maintenance.

In this context the protective layer has to sustain fire temperatures of up to 1350° Celsius and keep the temperature at the interface to the substrate and at the first reinforcement below specified threshold values, which on the one hand should be lower than the critical temperature of steel and on the other hand should prohibit constraint stresses in the structure (Figure 5). If this is not possible a complete fire design of the structure (Wageneder, 2006) at a reduced temperature level will be necessary. The bond to the substrate is loaded by the dead weight of the layer as well as compression/suction effects caused by traffic. The suction will increase with the speed of passing traffic and decreasing cross section of the tunnel. These effects require considerable high cycle fatigue (vibration) resistance of the material (Biennemann & Girnau, 2005; ZTV-ING, 2003; DB NETZ, 2002; EN 14067-3, 2003; EN 14067-5, 2006).

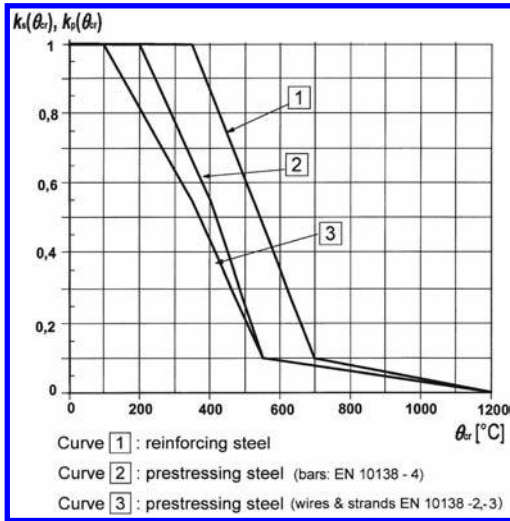


Figure 5. Reference curves for critical temperature of reinforcing steel θ_{cr} corresponding to the reduction factor $k_s(\theta_{cr}) = \sigma_{sf}/f_{yk}$ (20° Celsius) or $k_p(\theta_{cr}) = \sigma_{sf}/f_{pk}$ (20° Celsius) according to (EN 1992-1-2, Fig. 5.1, 2007), σ_{sf} steel stress in fire situation.

Water and de-icing salts in road tunnels will be conveyed onto the tunnel walls as a result of splashing by heavy traffic. During the year the temperature of the tunnel lining will typically fall below dew point several times resulting in condensation. Water penetrates even through the tunnel lining if no special measures have been taken. Usually this should not result in drip formation (OEVB, 2002; DAfStB, 2003). In many parts of the world frost attack will be possible in parts of the tunnel. This combined impact of water, frost or frost with de-icing salts should not reduce the lifecycle of the protective layer.

It is possible to replace protective layers after some years of service. But this should be avoided as far as possible due to the fact that our traffic tunnels need to operate to full capacity. In the references by Biennemann & Girnau (2005) and Haack (1994) it is suggested to replace protective layers 2 to 3 times over a 100 year service life. In Austria the specifications try to use only protective layers capable of 100 years of service life. One important advantage of protective layers is the possibility of replacing them easily after a fire disaster.

For the regular inspection of a tunnel for damage or changes in crack width it is beneficial to have a close look at the tunnel surface without any materials covering this surface. Especially for infrastructure which cannot be closed for inspection this work must be done in the most rapid way. Protective layers downgrade access to the structure. This can be resolved to some extent by “openings”

in the protective layer and by proper inspection before application. Cement-based layers normally show cracks arising or widening in the substrate, as they are brittle too. Materials with a very low modulus of elasticity have to prove that underlying cracks will be visible on their surface.

3.2 First experiences from preliminary tests

In the course of a pre-qualification trial for a tunnel project in Austria (Tunnel Lainz), several protective layers were tested. Some of the constitutive materials and some properties are given below.

The binders used were cements according to EN 197-1 (2008) as well as high-alumina cement. As low thermal conductivity will result in thin layer thickness many protective layers are produced with lightweight aggregates or with air entraining agents (or binders or aggregates which release chemically bound water, when heated). Therefore normal aggregates, lightweight aggregates (shell sand, vermiculite, perlite, aluminosilicate hollow spheres and expanded glass granulate), or high temperature resistant aggregates (e.g. Olivine) are used. Ceramic fibres, cellulose fibres and glass fibres are used alone or combined with micro-synthetic PP fibres.

Mixtures incorporating such constitutive materials result in mortars with a bulk density of 420 kg/m³ up to 2200 kg/m³, resulting in a required layer thickness of 25 mm to 80 mm (when a maximum temperature at the interface of ≤350° Celsius and at the first reinforcement ≤250° Celsius is called for). In comparison the bulk density of plate-like protective layers is between 600 kg/m³ and 900 kg/m³, resulting in a layer thickness for use in tunnels of between 18 mm und 30 mm. Due to the very different bulk densities of the products the compressive strength of these layers may vary from 3 MPa up to the strength classes of normal concrete.

First fire tests were performed at maximum temperatures of 1200° Celsius or 1350° Celsius. Some products performed well at 1200° Celsius, while they started melting at 1350° Celsius (Figure 6).

Modifications of some products became necessary after having tested the protective layers subsequently to hosing with water. Even protective

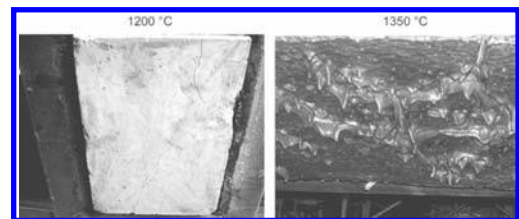


Figure 6. Example of a protective layer after a fire ingress of 1200 and 1350 degree Celsius.

layers can be damaged by explosive spalling under such conditions.

For overhead applications the owner required mesh reinforcement in addition to the bond of the material to the substrate. The use of reinforcement should help to avoid the formation of surface layers with inferior bonding that may loosen and endanger traffic passing by. The mesh should keep the protective layer in place until the next inspection.

For lightweight protective layers the mesh and the anchors have to be of stainless steel, for normal shotcrete ordinary reinforcement can be used. In any way it becomes clear that reinforcement and anchoring must be planned very carefully, as these are important safety and cost factors (Figure 7).

For inspection purposes it is essential to detect cracks and changes in cracks within the construction. Protective layers should not act as a crack bridging repair material. Tests were performed on slabs in bending (Figure 8). These tests demonstrated that cracks, which are in the substrate,

are reflected on the shotcrete surface too, even if micro-synthetic PP fibres are used.

To achieve good thermal insulation many protective layers are based on mixes with high porosity (up to 60%). This often results in high water absorption (coefficient of water absorption 0.10 to 0.90 kg/m²/h). If the layer is water-saturated, the risk of frost damage is very high. Up to now the degree of saturation of protective layers in tunnels is unknown.

Tests based on OENORM B 3303 (2002), without de-icing salt and with water on the tested surface (which is a very severe attack), showed frost damage material loss of up to 350 g/m² (the threshold value for concrete is usually 40 g/m²). Frost resistance is therefore a hot topic regarding lightweight protective layers in central Europe. Frost resistance in combination with de-icing salts has not been tested yet in this research.

3.3 Specifications in a technical bulletin of the Austrian Society for Concrete- and Construction Technology

The bulletin “Protective layers for the improved fire resistance of underground constructions” published by the Austrian Society for Concrete and Construction Technology (OEVBB, 2006), has to be applied for all mortars and panels which are used as protective layers in underground constructions made of concrete in Austria.

The producer of the material has to declare the main constitutive materials, the manner of application, the layer construction and the necessary layer thickness. Grading, bulk density, tensile strength and water content serve as identification.

The fire test is performed on two large-scale test specimens measuring 1800 × 1400 × 500 mm (L × W × H) made from normal-strength concrete (Figure 9). The protective layers are applied on the centre part of one side of the panels. The RWS



Figure 7. Example of mesh reinforcement together with anchorage for overhead application of shotcrete (picture: Ruzicka).



Figure 8. Bending test to demonstrate crack opening in protective layers (picture: Saxer).

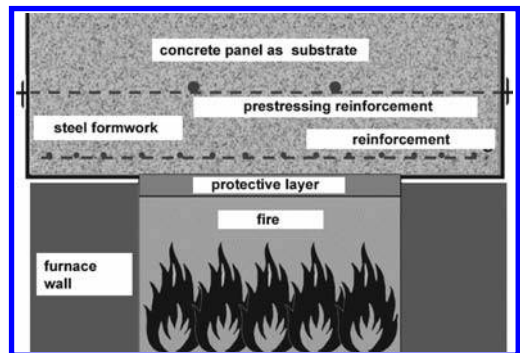


Figure 9. Cross section of a test specimen with protective layer on the oven.

time-temperature curve (Rijkswaterstaat) is used for the fire for 180 minutes (Tan, 1997). During the test it must be ensured that:

- the temperature at the interface to the concrete substrate does not exceed 350 degree Celsius,
- the temperature of the first layer of reinforcement in the concrete in 40 mm does not exceed 250 degree Celsius and
- no spalling occurs.

As there is no accepted testing procedure available for protective layers regarding their frost resistance, an existing testing procedure for concrete repair materials was used. The test for frost resistance with and without de-icing salts is always performed with de-icing salts using different threshold values. The test procedure follows the guideline “Maintenance and repair of structures made from concrete and reinforced concrete” (“Erhaltung und Instandsetzung von Bauten aus Beton und Stahlbeton”) of the Austrian Society for Concrete- and Construction Technology (OEVB 2003, Appendix 8, “Frost action with de-icing salts” (“Frost-Taumittel-Beanspruchung”).

The requirements for maximum allowable decrease in mass when damage takes place at the surface of the specimen after 56 cycles are summed up in Table 1.

The amount of lost material estimated as mass per testing area is calculated as volume loss using the bulk density of the material. Thereby it is

Table 1. Requirements for protective layers regarding frost resistance (Bulletin “Protective layers for the improved fire resistance of underground constructions” published by the Austrian Society for Concrete- and Construction Technology) The requirements differ by exposure classes according to EN 206-1 (2005). XF is an European exposure class for freeze/thaw attack, 4 standing for the most severe attack. For panels the last column does not apply.

Requirement category	Decrease in mass cm^3/m^2 after 56 frost-cycles ¹	Bond strength in comparison to 28 day value
XF 4	≤ 70	$\geq 70\%$
XF 2	≤ 300	$\geq 60\%$
XF 1 = XF 3	≤ 600	$\geq 50\%$

¹ max. depth of material loss shall not exceed 1 mm (XF 2) or 4 mm (XF 1) and the loss in volume shall not increase with time ($V_{42-56 \text{ FC}} \leq V_{28-42 \text{ FC}}$).

XF 1 = XF 3 for road tunnels with coatings and all railway tunnels with frost attack.

XF 2 for road tunnels with coating near the portal.

XF 2 for road tunnels without coating with frost attack.

XF 4 for road tunnels without coating near the portal.

Table 2. Requirements regarding bond strength versus bulk density of the protective layer (bulletin “Protective layers for the improved fire resistance of underground constructions” published by the Austrian Society for Concrete and Construction Technology OEVB (2006)).

Oven-dry density ρ in kg/m^3	Bond strength f_b in MPa
$\rho > 2000$	$f_b \geq 1.5$
$2000 > \rho > 1000$	linear reduced from 1.5 to 0.4
$1000 > \rho > 400$	linear reduced from 0.4 to 0.2
$\rho < 400$	$f_b \geq 0.2$

possible to compare the results of protective layers with different densities. As the toughest testing procedure is selected for all exposure classes, the threshold values regarding material loss are much higher for the exposure classes XF 1 and XF 3. The advantage would be that all materials can be tested with the same procedure. Practice will show if this is a suitable way of testing. In any case this testing procedure cannot be applied for layers consisting of materials which do not follow the same deterioration processes as concrete and lightweight concrete (e.g. delamination, swelling, upraising of fibres). In this case other testing procedures have to be applied (e.g. Fagerlund, 1977; Brameshuber et al. 2005).

The application of barriers onto the substrate of an existing tunnel requires preparation of the surface, normally a roughness equal to more than 1 mm, and bond strength that may vary with the bulk density of the protective layer (Table 2). “When applying shotcrete or gunite or any other lightweight mortar, mesh reinforcement for all overhead applications is required besides the proven bond strength. This mesh, together with anchors, has to be designed in that way that all regular loads during the life time do not lead to a local breakdown of the layer” (OEVB, 2006).

For this anchoring system the following requirements are essential (condensed text from OEVB 2006):

- The anchors have to keep the layer in place together with the mesh reinforcement without using the bond of the layer itself. Loads resulting from the dead weight of the layer and the maximal forces from suction have to be taken into account. The anchors have to be approved for use with the specific substrate. The partial safety factor for the action is $\gamma_s = 1.5$, for the anchor the partial safety factor for resistance fixed in the approval for a single anchor must be chosen.
- Resistance against corrosion must be proven.

- The mesh reinforcement must be fixed force-locked to the anchors. The mesh should not vibrate when embedded by shotcrete. Drilling the anchor holes must not damage the substrate or embedded reinforcement.

Plate-like protective layers are in contrast subjected dynamically to alternating stresses of suction and pressure caused by passing traffic over long periods. Their fastening to the lining must take the fatigue of the anchors into account.

The detection of cracks in the protective layer due to cracks in the substrate is carried out by flexural bending tests on two panels measuring $3.60\text{ m} \times 0.80\text{ m} \times 0.6\text{ m}$, made from concrete C 25/30. Previous to the application of the protective layer both panels are cracked to 0.5 mm crack width by four-point loading. The cracks are marked and the panels unloaded. At the age of 28 days the protective layer (mortar or shotcrete) is applied together with the mesh reinforcement and anchorage with a length of 2.8 m between the supports. The shotcrete is cured and stored for a further 28 days. Then the bonded panel is tested and checked for deformation in a flexural bending test using a span of 3.0 m. The crack formation versus deflection in the substrate panel and the protective layer is observed continuously.

It must be demonstrated that a crack opening or crack widening of $\Delta w > 0.5\text{ mm}$ in the panel leads to visible cracks at the surface of the protective layer. For cement mortars and shotcrete with bulk density of $\rho > 1000\text{ kg/m}^3$ and bond strength $> 0.4\text{ MPa}$ no test is required, as the preliminary tests clearly demonstrated a positive result. For mineral mortars with a bulk density between 300 and 1000 kg/m^3 specific agreements are suggested. Quality assurance matters together with the required tests are also addressed in detail in this bulletin.

3.4 Availability and cost structure

Sprayable protective layers are supplied by the producers of fire protection systems as well as premix producers and as polypropylene-fibre-reinforced shotcrete by ready-mix concrete plants. The application is performed by companies active in fire protection, but also by construction companies specializing in concrete repair work. The cost consists of the following elements:

- Substrate preparation,
- Mesh reinforcement and anchorage,
- Material and application,
- Site set-up and scaffolding.

As application in tunnels is quite new, only estimated costs can be given. Substrate preparation, depending on the site situation, without site

set-up costs for undamaged concrete, is in the range 8 €/m^2 to 12 €/m^2 . Lightweight gunite or polypropylene fibre reinforced shotcrete applied to the required thickness may be estimated vertically applied at between 90 €/m^2 and 120 €/m^2 ; overhead application will raise the cost to 100 €/m^2 to 140 €/m^2 (Kusterle et al. 2006; Kusterle & Vogl, 2008) including application, reinforcement and anchors. Plate-like protective layers (construction boards) are provided by a limited number of companies. Costs are comprised of:

- Construction boards
- Anchorage
- Site set-up and scaffolding.

Evenness of substrate surface, orientation and structuring as well as required tolerances at joints will influence the costs. For the board $50\text{--}70\text{ €/m}^2$ will have to be spent, depending on the type and thickness. For anchors $15\text{--}20\text{ €/m}^2$ and for attachment about 45 €/m^2 have to be calculated. Site set-up and scaffolding may vary depending on the project.

4 APPLICATION OF PP-FIBRE-REINFORCED SHOTCRETE AS A PASSIVE PROTECTIVE LAYER IN THE TUNNEL LAINZ

4.1 Project Tunnel Lainz

The Tunnel Lainz is part of the new railway “entrance” to Vienna from the west. The part of the tunnel addressed here (LT22-LT25), was built using the cut-and-cover method (Vogl et al. 2006, Vogl 2007, Kusterle & Vogl, 2008).

The line passes built-up areas, crossing streets and rivers. An existing line is partly situated on top of the planned tunnel. Therefore an adequate passive fire protection of the structure is essential. In 2004 the Austrian Railway authorities (HL-AG, ÖBB-Infrastruktur Bau AG since 2005) issued an invitation to tender for covering the existing concrete structure with fire protective layers. The length of the tunnel concerned was about 1800 m, the total area about $43,000\text{ m}^2$. Construction boards were applied in an area of $13,000\text{ m}^2$, but most of the work was done with polypropylene-fibre-reinforced shotcrete.

Most of the shotcrete (about $23,000\text{ m}^2$) was applied in the LT23 section, where the tunnel has a span of 24 m and a height of 12 m. The slab beams in this area have a hammer-head cross-section, which made the application more difficult (Figure 10). To meet this challenge a specially developed control system was needed for the robot which was used for the spraying application.



Figure 10. Crossover in the Tunnel Lainz, LT 23, Austrian railways (picture: Vogl).

4.2 Substrate preparation

The preparation of the substrate prior to the shotcrete application was done by high pressure water jet at 2500 bar from special scaffolding. A bond strength of ≥ 1.5 MPa and a roughness equivalent of ≥ 1.0 mm had to be achieved.

4.3 Polypropylene-fibre-reinforced shotcrete

The tender for the application of protective layers was the first for such a tunnel in Austria. The use of lightweight mortars was possible as well as the use of shotcrete with polypropylene fibres. Both had to meet tough requirements regarding frost action (the bulletin of the OEVB (2006) was not published then, resulting in somewhat different requirements). Lightweight protective layers were not able to fulfil the requirements in the short term.

Therefore shotcrete with a layer thickness of 80 mm was envisaged for most of the work. Only the ceiling between the beams was covered to a thickness of 60 mm. If a lightweight mortar could have been used the thickness could have been reduced to 40 mm.

For safety reasons a mesh reinforcement was included as part of the protective layer. Due to dead weight and suction, together with the safety factor, loads of 8.0 kN/m^2 must be carried. Anchors tested for use in cracked concrete had to be used. When drilling the anchor holes it was not permitted to damage the existing reinforcement. The anchors should only be loaded by tension loads. Even the connection between anchor and mesh had to be designed for the applied load. Approval was received following the performance of special tests on the individual members and an enhanced testing programme.

The fibre-reinforced shotcrete complied with the Austrian class FRSpC 20/25/III/BB2G/HZ1,5

(Novimontan produced by Schretter & Cie, strength class 20/25, class III for permanent use, fire resistance class BB2G, bond strength >1.5 MPa). Polypropylene fibres with a diameter of $18 \mu\text{m}$ and 6 mm length were used at a dosage of 2.0 kg/m^3 . The fibre had been tested according to the Guideline for Fibre Reinforced Concrete by the Austrian Society for Concrete and Construction Technology using an RWS fire (OEVB, 2008). This product was chosen after several trials with reference to the throughput, surface appearance and application by robot. The dry-mix process was used. A control-chamber shotcrete machine was used as a shotcrete gun. About 20 to 30 tonnes of dry mix were sprayed during one shift. Compressive strength reached 31 MPa. Water penetration tested in accordance with Austrian standard B 3303 (OENORM, 2002) was 25 mm. Testing bond strength was somewhat difficult, as the layer thickness was 60 to 70 mm and reinforcement was present in the layer and the substrate. The required 1.5 MPa could always be reached.

4.4 Shotcrete robot

When preparing this work it became clear that the process had to be automated in some way. For an automated shotcrete application a robot was the first choice. Smooth surface, constant cross sections, high shotcrete throughput and uniform quality were the crucial factors. Advantages in the construction sequence and good experience with the automation of high water pressure jet processes for concrete demolition were other decisive factors.

The catalogue of requirements was only met by one robot from MEYCO, Type Logica 15. This system allows scanning of the substrate by a laser and subsequent fully automated application of the shotcrete. Overriding the process is possible.

In cooperation with the producer of the system a full scale model of the hammer-head beams was set up in the producers' premises. This model was used to test the software and optimize the rows and row gaps followed by the nozzle. The scaffolding was designed following the robot producer's requirements to provide a rigid platform. A stiff platform was necessary to achieve exact rows of sprayed concrete (Vogl et al. 2006).

The main problem was to find a way of spraying the hammer-head beams with the robot using an automated process. Due to the geometry of the cross section, the operation of the laser scanner was limited. Therefore this method became uneconomical. For use with the Logica system the robot was upgraded by angle and distance sensors. With eight degrees of freedom, the robot can be used in manual, semi-automatic or fully automatic mode.

For this special application a programme was adapted which is similar to the software used with machine tools. The starting point of the control system was the exact dimensions of the application areas and the correlating angles. The starting point, angularity, and nozzle distance as well as kinematics have to be defined on the laptop using this software. In the course of trial applications the nozzle feed rate, the shotcrete throughput and the nozzle oscillation had to be adjusted for final optimization of the shotcrete (Figure 11).

When shotcreting in the automatic mode the nozzle head moves in rows with a constant speed over the receiving substrate. By adjusting the speed and row gap the required layer thickness is achieved. Other parameters influencing the layer thickness, such as throughput, nozzle distance and nozzle oscillation, are presumed to be constant. The application is done automatically in the area of the scanned 3-dimensional grid, by measuring the distance from the laser head to the receiving surface. Using the remote control, it is possible to override the automatic spraying at any time. Following use of the manual mode the process will continue automatically.

4.5 Experiences from this application

For the job at the Tunnel Lainz LT22-LT25 the 30,000 m² of PP-fibre-reinforced shotcrete application was successfully realized between July 2005 and September 2006. Different problems had to be solved when applying PP-fibre-reinforced shotcrete for the first time (Vogl, 2007). The problems were intensified by the short construction time, the limited space and the complicated geometry of most of the receiving substrate. PP-fibre-reinforced dry mix shotcrete proved to be difficult to handle. The following problems occurred:

- Blocking of the dry mix when filling the silos,
- increased wear and tear of equipment,
- reduction in throughput by more than 50%,
- hampered working conditions,
- damage of equipment due to blocked filter and intake by fibres (Vogl, 2007).

As this was the first application of this type of protective layer, a dense control and test programme was necessary. The use of PP microfibres resulted in adaptations in the production process, especially the silo, the shotcrete gun and the application. Due to the close cooperation of all the people involved, the final result was nearly perfect. Regarding durability and quality, use of a micro-synthetic PP-fibre-reinforced shotcrete as a fire protective layer is in the contractor's view the optimum solution. But there is still quite a big potential for improvements. In future the wet mix process will be favoured, as less fibres will be lost in the overspray with this method. In recent years the following areas of tunnel linings have been covered by protective layers in Austria:

- PP-fibre-reinforced shotcrete 63,000 m²,
- lightweight mortar about 20,000 m²,
- construction boards 220,000 m².

In the same period 38 km of new tunnels were built using micro-synthetic PP-fibre-reinforced inner linings in Austria.

5 SUMMARY

Currently the following advantages may be listed for the use of sprayable protective layers (PP-fibre-reinforced shotcrete, sprayable lightweight mortars) in comparison to other measures (e.g. micro-synthetic PP-fibre-reinforced concrete inner lining).

The advantages include: thermal isolation of the structure; excellent fire protection, which can be tailored by adapting the layer thickness; longer fire resistance than with other possible methods; application on any geometrical form of receiving surface; excellent durability in the case of PP-fibre-reinforced shotcrete; fast and easy rehabilitation subsequent to a fire disaster; no special concrete or reinforcement necessary for the tunnel lining; existing tunnels with durability problems can be upgraded by increasing concrete cover (this additional value requires a dense normal weight shotcrete); simple bond Quality Control by knocking with a hammer on the surface and by mapping cracks. The disadvantages include: for lightweight mortars it is difficult to prove the same durability as concrete inner linings; no direct access to the structure; profile reduction; surface treatment and



Figure 11. Shotcrete application at the hammer-head beams in the Tunnel Lainz, LT 23, Austrian railways (picture: Vogl).

anchoring is time-consuming and costly; complex site set-up and high cleaning expense.

Plate-like protective layers (construction boards) differ in the following aspects from sprayed products: generally they need no substrate preparation except local deburring; dry, mortarless construction without any overspray from shotcrete work; boards are easily replaced if local damage occurs; smooth surface of prefabricated boards but curvature of the tunnel, keeping straight joints within small tolerances and inspection openings as well as mounting parts increase application costs; high cycle fatigue strength requirements coming from alternating stresses due to suction and pressure lead to a closer pattern of anchorage than in housing; cracks and defects are hidden for a long time.

6 CONCLUSION

Due to recent fire disasters passive fire protection in traffic tunnels is a hot topic. One of the possible measures for improving fire resistance, especially reducing explosive spalling, is the application of protective layers which thermally isolate the structure from fire ingress. The bulletin "Protective layers for the improved fire resistance of underground constructions" published by the Austrian Society for Concrete and Construction Technology gives for the first time guidance regarding the application of protective layers. Shotcrete reinforced by polypropylene microfibres is an excellent material for this purpose. The first lightweight sprayable mortars are also available for tunnel application. But frost and durability aspects are still under development for this type of material. Their superior insulation properties result in thinner layers than for shotcrete.

REFERENCES

- Biennemann, F. & Girnau, G. (ed.) 2005. *Brandschutz in Fahrzeugen und Tunneln des ÖPNV*. Verband Deutscher Verkehrsunternehmen, Stuva, Alba Fachverlag, Düsseldorf.
- Bramshuber, W., Pierkes, R., Tauscher, F. & Friebe, W.-D. 2005. Anrechnung von Flugasche bei Betonen für Innenschalen von Strassentunneln. *beton* 7+8.
- DAfStB 2003. *Watertight concrete constructions* (Wasserundurchlässige Bauwerke aus Beton (WU-Richtlinie)). Berlin.
- DB NETZ 2002. *Deutsche Bahn Richtlinie 853: Eisenbahntunnel planen, bauen und instand halten—Modul 853.2001 Standsicherheitsuntersuchungen*.
- EN 197-1 2008. *Cement—Part 1: Composition, specifications and conformity criteria for common cements (consolidated version)*.
- EN 206-1 2005. *Concrete—Part 1: Specification performance, production and conformity (consolidated version)*.
- EN 1992-1-2 2007. Eurocode 2: Design of concrete structures—Part 1-2: General rules—Structural fire design (Planung von Stahlbeton- und Spannbetontragwerken—Teil 1-2: Allgemeine Regeln—Tragwerksbemessung für den Brandfall).
- EN 14067-3 2003. *Railway applications—Aerodynamics—Part 3: Aerodynamics in tunnels* (Bahnanwendungen—Aerodynamik Teil 3: Aerodynamik im Tunnel).
- EN 14067-5 2006. *Railway applications—Aerodynamics—Part 5: Requirements and test procedures for aerodynamics in tunnels* (Bahnanwendungen—Aerodynamik Teil 5: Anforderungen und Prüfverfahren für Aerodynamik im Tunnel).
- Fagerlund, G. 1977. The Critical Degree of Saturation Method of Assessing the Freeze/thaw Resistance of Concrete. *Matériaux et Constructions*, RILEM, 10 (1977) No. 58, pp 217 ff.
- Florian, A. 2002. *Schädigung von Beton bei Tunnelbränden*. Diploma-thesis, University Innsbruck.
- Haack, A. 1994. Nachträgliche Brandschutzmassnahmen in Tunneln—Technische und wirtschaftliche Gesichtspunkte. In: Stuva (ed.) *Forschung + Praxis, Ö-Verkehr und Unterirdisches Bauen*, Volume 35, Cologne, Alba Verlag, Düsseldorf.
- Hertz, K., D. 2003. Limits of Spalling of Fire-Exposed Concrete. *Fire Safety Journal* 38.
- Kalifa, P., Menneteau, F.-D. & Quenard, D. 2000. Spalling and Pore Pressure in HPC at High Temperatures. *Cement and Concrete Research* 30.
- Kalifa, P., Chene, G. & Galle, C. 2001. High temperature behaviour of HPC with polypropylene fibres: from spalling to microstructure. *Cement and Concrete Research* 31.
- Kordina, K. & Meyer-Ottens, C. 1981 and 1999. *Beton Brandschutz Handbuch*. Düsseldorf: Beton-Verlag GmbH.
- Kusterle, W., Lindlbauer, W., Hampejs, G., Heel, A., Donauer, P.-F., Zeiml, M. et al. 2004. Fire Resistance of Fibre-reinforced, Reinforced, and Pre-Stressed Concrete (Brandbeständigkeit von Faser-, Stahl- und Spannbeton). In: Austrian Federal Ministry of Transportation, Infrastructure and Technology (ed.), (Bundesministerium für Verkehr, Innovation und Technologie), *Road research report 544* (Strassenforschung Heft 544), Österreichische Forschungsgemeinschaft Strasse und Verkehr, www.fsv.at, Vienna.
- Kusterle, W., Ruzicka, M., Donauer, P.-F. & Muchsel, H. 2006. Fire Protective Layers in Traffic Tunnels. (Brandschutzschichten in Verkehrstunneln). In: Kusterle, W. (ed.) *Spritzbeton Technology 06*, University Innsbruck.
- Kusterle, W. & Vogl, G. 2008. Brandschutzschichten in Verkehrstunneln. *beton* 3/2008.
- Meyer-Ottens, C. 1972. *Zur Frage der Abplatzungen an Betonbauteilen aus Normalbeton bei Brandbeanspruchung*. Thesis, University Braunschweig.
- OENORM B 3303 2002: *Testing of Concrete (Betonprüfung)*. Vienna.
- OEVB (Austrian Society for Concrete- and Construction Technology) 2002. *Guideline for watertight concrete*

- constructions (Richtlinie Wasserundurchlässige Betonbauwerke- Weisse Wannen). Vienna.
- OEVB (Austrian Society for Concrete- and Construction Technology) 2003. *Guideline for Maintenance and Repair of Structures Made from Concrete and Reinforced Concrete* (Richtlinie Erhaltung und Instandsetzung von Bauten aus Beton und Stahlbeton). Vienna.
- OEVB (Austrian Society for Concrete- and Construction Technology) (ed.) 2004. *Guideline for Shotcrete* (Richtlinie Spritzbeton). Vienna.
- OEVB (Austrian Society for Concrete- and Construction Technology) 2005: *Guideline for Improved Fire Protection of Underground Constructions by Concrete*. (Richtlinie Erhöht brandbeständiger Beton). Vienna.
- OEVB (Austrian Society for Concrete- and Construction Technology) 2006. *Bulletin Protective layers for the improved fire resistance of underground constructions* (Merkblatt Schutzschichten für den erhöhten Brandschutz unterirdischer Verkehrsbauwerke). Vienna.
- OEVB (Austrian Society for Concrete- and Construction Technology) 2008. *Guideline for Fibre Reinforced Concrete* (Richtlinie Faserbeton). Vienna.
- Paliga, K. 2003. Ursachen von Betonabplatzungen bei Tunnel bränden und Schutzmassnahmen. In: Institut für Baustoffe, Massivbau und Brandschutz (ed.) *Braunschweiger Brandschutz Tage '03*, vol. 168, Braunschweig.
- Persson, B. 2006. *Mitigation of explosive temperature spalling of self compacting concrete with fibres*. RILEM TC DSC report.
- Schneider, U., Diederichs, U. & Horvath, J. 2001. Zum Abplatzverhalten von Hochleistungsbetonen unter Brandangriff. In: Technical University Vienna (ed.), *Schriftenreihe des Institut für Baustofflehre, Bauphysik und Brandschutz*, vol. 7; Vienna.
- Schneider, U. & Horvath, J. 2002. Abplatzverhalten von Tunnelinnenschalenbeton unter hohen Temperaturen. *Zement und Beton*, Volume 1, 2002, p 10–14, Vienna.
- Tan, G.L. 1997. Fire Protection in Tunnels Open to Hazardous Goods Transport, Experience in the Netherlands. *La Securite Dans Les Tunnels Routiers*, Paris.
- Tatnall, P. C. 2002. Fibre Reinforced Sprayed Concrete: The Effect on Anti-Spalling Behaviour during Fires. In Fourth International Symposium on Sprayed Concrete—*Modern Use of Wet Mix Sprayed Concrete for Underground Support*, Davos.
- Ulm, F.-J., Coussy, O. & Bazant, Z. 1999. The “Chunnel” fire: Chemoplastic Softening in Rapidly Heated Concrete. *Journal of Engineering Mechanics* 125.
- Vogl, G.; Tatzl, B. & Mayr, R. 2006. Spritzbeton bei Tunnelinstandsetzung und als Brandschutzschicht mit Einsatz von Spritzrobotern. In: Kusterle, W. (ed.) *Spritzbeton-Technologie 2006*, University Innsbruck.
- Vogl, G. 2007. Baulicher Brandschutz in Strassenverkehrsbauten mit Faserspritzbeton. *Zement und Beton*, Vienna.
- Wageneder, J. 2006. *Tragverhalten von Stahlbetontragwerken unter Hochtemperatureinfluss*. Doctor-thesis TU-Graz.
- Waubke, N. V. 1973. Über einen Physikalischen Gesichtspunkt der Festigkeitsverluste von Portlandzementbetonen bei Temperaturen bis 1000°C. Brandverhalten von Bauteilen. In: Deutschen Forschungsgemeinschaft (ed.), *Schriftenreihe des Sonderforschungsberichts 148*, Volume 2, TU Braunschweig.
- Wetzig, V. 2000a. *Destruction Mechanisms in Concrete Material in Case of Fire, and Protection Systems*. Hagerbach Test Gallery Ltd., Switzerland.
- Wetzig, V. 2000b. Beton mit erhöhter Brandbeständigkeit. *Schweizer Ingenieur und Architekt* 43.
- Wille, K.; Dehn, F. & Kützing, L. 2003. Fire Resistance of High Strength Concrete (HSC) Columns with Fibre Cocktails under loading. In: University Leibzig, *Annual Civil Engineering Report*, No. 8, Leipzig.
- ZTV-ING 2003: Zusätzliche Technische Vertragsbedingungen und Richtlinien für Ingenieurbauwerke—Teil 5 Tunnelbau. Bundesministerium für Verkehr, Bau- und Wohnungswesen (Federal Ministry of Transport, Building and Urban Affairs, Germany), Verkehrsblatt-Sammlung 1056.

Shrinkage and durability of shotcrete

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ABSTRACT: Cracking of shotcrete in tunnels has in recent years become a problem in Sweden. Thus an investigation was initiated. The results show that shotcrete differs from ordinary cast concrete in many ways. Basically, wet shotcrete is ordinary concrete but is mixed with a set accelerator at the nozzle and as a result the major cement reactions will take place in a stiff (but not hardened) material. Cement hydration results in shrinkage that in ordinary concrete will be compensated for due to settlement, but in the stiffened shotcrete it will change the properties of the concrete. It will give a different structure and mode of porosity, which in turn will increase shrinkage. Especially with an alkali free set accelerator the shotcrete develops a larger amount and coarser porosity, which in turn will generate significantly larger drying shrinkage than in cast concrete. Moreover, the shotcrete is very sensitive to early shrinkage if not properly water cured. Water must be added to compensate for the chemical shrinkage in the stiff but not hard structure of the young shotcrete.

1 INTRODUCTION

In Sweden, especially in recent years, there has been problem with cracking of shotcrete in tunnels most of which has been constructed using wet-process shotcrete. This is especially the case where the shotcrete is covering drainage systems, where the bonding to the substrate is weak. The purpose of this investigation was to determine if shotcrete exhibits a greater tendency to shrink and crack than ordinary cast concrete. Basically the shotcrete is based on ordinary concrete but there are differences.

A shotcrete mix normally has a smaller maximum aggregate diameter $D_{\max}$ and contains more cement that will increase shrinkage but the major difference relative to ordinary cast concrete is that shotcrete is mixed with a set accelerator at the nozzle and is stiffened very quickly. Moreover, in later years water glass has been replaced with an alkali-free (aluminum sulphate salt) type set accelerator, another variable that will change the properties.

In shotcrete the early cement hydration reactions will take place in a stiff structure produced by the set accelerator. Thus the shotcrete can not adjust to early volume changes in the way that normal cast concrete can. Moreover, the set accelerator will interact with cement hydration. This will generate an altered structure within the binder and thus one can not assume that shrinkage will be the same as in cast concrete. It will also depend on the type of set accelerator as the different types

will interact with the cement hydration in different ways and result in different internal hydration structures. This paper will include a theoretical discussion and some laboratory tests to examine the relationship between shotcrete, binder reactions, binder structure, and shrinkage.

1.1 Shrinkage

All concretes shrink but this is a complex process that is influenced by several factors, see Figure 1. Basically there are two types of shrinkage: autoge-

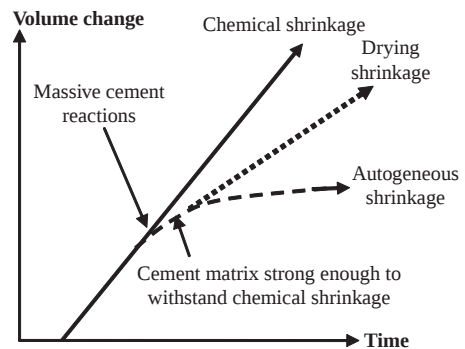


Figure 1. Sketch illustrating the different types of shrinkage. Chemical shrinkage is a theoretical value, autogenous free shrinkage is the measured value and drying shrinkage occurs when the matrix is stiff.

neous shrinkage and drying shrinkage. These occur as a result of chemical reactions, development of the cement paste, and evaporation of water. Different causes will affect the concrete at different stages. Thermal volume changes that also can result in cracks are not considered in this paper.

In the young fresh concrete shrinkage is mainly linked to cement hydration. The volume of water and cement is larger than that of the cement hydrates, which will create shrinkage forces as long as cement is hydrating. This is called chemical shrinkage and is the force behind autogeneous shrinkage. In a fully hydrated paste with a w/c of 0.4 the volume change is around 7–8 volume % depending on type of cement. The fluid and plastic concrete will contract but this will change when the concrete hardens. Moreover, the stiff but not hardened shotcrete will have a large open surface and will thus be sensitive to water movements. When the concrete is strong enough to withstand stresses due to chemical shrinkage, this will first result in an internal under pressure and later in an increased porosity. Thus the autogeneous shrinkage is smaller than the chemical shrinkage. With cement paste and a water/cement ratio of 0.4 and CEM I cement the autogeneous shrinkage will be around 0.5 vol. % (Justnes et al. 1999, Hammer 1999). This is equivalent to a one dimensional free shrinkage strain of around 1.6‰. The bulk of the concrete is made up of aggregate and if we assume 75 vol. percent aggregate this would result in a shrinkage strain of 0.4‰.

In ordinary fluid concrete this is of subordinate importance but it will affect the shotcrete as at this stage the early hydration will take part in a stiff structure. Added water will compensate for the chemical shrinkage. Thus one can presume a larger shrinkage of shotcrete if water is not added. If the concrete loses water during the plastic stage one can get plastic shrinkage that results in wide and deep cracks.

In a properly hardened concrete one will mainly get drying shrinkage that is a result of evaporation of water from the capillary system. When the capillary pores are drying out they will contract and the concrete will shrink. Changes of the pore structure will affect the amount of drying shrinkage and one can not presume that shotcrete has the same porosity as ordinary cast concrete, both due to the application technique that gives a more inhomogeneous product and chemical shrinkage in a stiff structure.

The consequences of shrinkage depend on several parameters. The shrinking forces may result in increased porosity, micro cracks, internal cracks or free shrinkage. In the laboratory free shrinkage can be measured. In most standard tests free shrinkage of hardened concrete is measured over time. As the shrinkage is not restrained, large cracks do not normally form but a part of the shrinkage may result in micro-cracks. When the shrinkage results

in internal cracks the free shrinkage will be less. Formation of cracks instead of free shrinkage will in most cases be the result of internal restraint due to reinforcement, aggregates, fibres etc. External restraint leads to wide cracks. Measurements on fresh cast concrete with a w/c ratio of 0.4 gave an autogeneous shrinkage of 0.25‰ (Holt & Leivo 1999) which is less than predicted from paste experiments. This is due to the internal restraint provided mainly by the larger aggregate particles.

2 CHEMISTRY AND FUNCTION OF SET ACCELERATORS

Wet shotcrete is ordinary concrete that is mixed with a set accelerator in the nozzle. In the normal case ordinary Portland cement and local aggregates are used. It must ordinarily display good pumpability and be formulated so that the rebound is small. To get this the $D_{\max}$ is smaller than in ordinary cast concrete and the amount of cement is higher. In Sweden the concrete mix will often contain 5 wt. % silica fume (by weight of cement) to make the concrete more cohesive. Silica fume is known to increase autogeneous shrinkage.

Set accelerators are chemicals that react with the early cement hydrates and alter the pore solution. The reaction forms either a network of solid products or, in the case of water glass, a viscous gel that stiffens the concrete and keeps it in place. The reaction must be fast and starts already in the air before the concrete reaches the intended surface. If it is too fast there will be rebound and if too slow the concrete will slough or fall out. Thus the amount needed for a certain thickness must be carefully adjusted.

2.1 Background to cement chemistry

When in contact with water, cement reacts exothermically and forms cement hydrates. The cement hydrates are in equilibrium with the water, called the pore solution. The reactions are slow in the beginning (during the dormant period) but after a certain time rapid cement reactions start (acceleration period) followed by slower reactions that increase the strength. During the dormant period the concrete can be cast and during the acceleration period the concrete first stiffens and then hardens. The different periods can be identified by isothermal calorimetry that measures the heat generated by the exothermic cement hydration reactions.

Cement clinker includes four main minerals: alite (C_3S), belite (C_2S), calcium aluminate (C_3A) and ferrite (C_4AF), where $C = CaO$, $S = SiO_2$, $A = Al_2O_3$, $F = Fe_2O_3$ and $H = H_2O$. In addition the cement contains calcium sulphate (gypsum) to regulate the properties of the fresh concrete. Alite and belite are

the main clinker minerals that form the cement gel, the calcium silicate hydrate (C-S-H) and the portlandite (CH). The calcium aluminate is a very reactive clinker mineral that promote the onset of hydration. Without gypsum it would lead to “flash setting” by forming calcium aluminate hydrates. Gypsum (calcium sulphate) reacts with aluminates and forms ettringite. The same mineral is formed with the introduction of aluminum sulphate set accelerators (see Section 2.2.2). Gypsum-free cement leading to flash setting is sometimes used in dry shotcrete.

The set accelerators react during the dormant period with the components of the pore solution. During the dormant period the normal cement reactions are stifled by the early cement hydrates. The pore solution is dominated by Ca^{2+} , Na^+ , K^+ , OH^- and SO_4^{2-} ions (Taylor 1997) and the set accelerator must react with one or several of these ions. During the acceleration period, massive cement reactions starts. In the hardened concrete the contents of calcium and sulphate ions are low and the pore solution is dominated by alkali hydroxides. The length of the dormant period depends on the type of cement and the temperature. The speed of setting and hardening depends on cement fineness and temperature. The rock walls in tunnels are normally relatively cool and thus the onset of the acceleration period is often late.

In Sweden, a low alkali, sulphate-resistant and slow reacting cement (Anläggningcement from Cements AB, CEM I, MH, SR, LA) is demanded by several authorities and is thus commonly used and was thus used in this investigation. It is rather coarse grained, high in C_2S and low in C_3A . With this type of cement and commonly cold tunnel walls it can take several days until the massive cement hydration and proper hardening starts. This will probably result in higher dosages of set accelerator and thus increase the shrinkage problems.

Ordinary accelerators used in cast concrete regulate the dormant period and/or the speed of reactions during the acceleration period. They are not effective enough to be used in shotcrete. Thus, in shotcrete another group of accelerators called set accelerators have been developed and used. The set accelerators are not supposed to interfere with the major cement reactions. They will react with the pore solutions of the young cement paste during the dormant period. This is basically a false or flash set generating a structure within which the rapid hydration of the acceleration period takes place, resulting in proper hardening.

2.2 Set accelerators and their reactions with Portland cement

The intension with the set accelerator is to make the concrete stiffen very quickly. To do this the set

accelerator must react with components of the pore solution and form a product that is stiff enough to keep the shotcrete in place. The set accelerator will interact with the cement hydration during the dormant period. Thus, it must react with the pore fluids formed during the dormant period. During this period the pore solutions are dominated by calcium and sulphate ions. In cement paste there are two major chemical systems, the silicate and aluminum systems. The set accelerator will interfere in this system and forms either calcium-silicate-hydrate or ettringite that are products of normal cement hydration. Sometimes the set accelerators also contain some alkali carbonate that precipitates calcite, which also leads to a stiffening action.

There are three principal types of set accelerators available:

1. Water glass (alkali silicate),
2. Alkali accelerators,
3. Alkali-free accelerators.

Of these three, the alkali-free accelerator type currently dominates the market. A cheaper product, called ‘water glass’, was more common earlier, but it leads to an alkaline fog that may harm workers. The alkali free accelerator allows thicker concrete layers to be sprayed and is thus more cost effective in production.

When the concrete is fluid, during the dormant period, the bulk of the cement is still unhydrated. The set accelerator will change the composition of the pore fluid but in general the hydration of the cement will occur independently of the set accelerator. This means that during the initial period the early strength is wholly dependent on the structure of set accelerator products (Figure 2). The basic prerequisite for the stiffening of water glass and alkali-free accelerators is enough of Ca in the pore solution (see later). The release of Ca ions will be somewhat slower with the Swedish cement compared to normal “standard” cement but the reactions will take place. The later strength will be due to

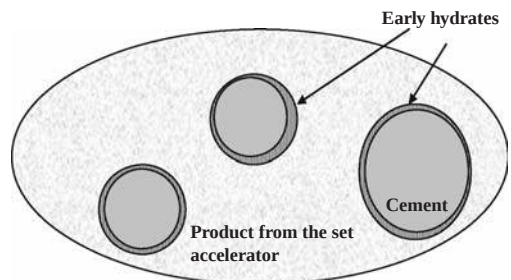
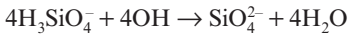


Figure 2. The set accelerator will form a loose structure between the cement grains. In this structure the proper hydration will eventually occur.

the cement hydrating in a structure affected by the set accelerator. Thus the composite structure will not be the same as in an ordinary cast concrete.

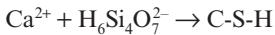
2.2.1 Water glass

Water glass is an alkali silicate in solution. Earlier it was made from quartz that was dissolved in warm concentrated alkali hydroxide. Water glass has a high pH, between 11 and 12 depending on composition. The solubility of silica increases with the pH. Stepwise the silica complex loses its protons with increasing pH.



This will lead to a successively higher negative charge on the surfaces. In an aqueous solution in equilibrium with silica at high pH, ionized polymers like $\text{H}_6\text{Si}_4\text{O}_7^{2-}$ can also be found (Dove & Rimstedt, 1994).

With the presence of monovalent cations (alkali) ions in water glass, the silica ions and silica-ion-complexes stay in solution. In the case of divalent ions like calcium, the silica ions will, due to the strong negative surface charge, bond to calcium and cause precipitation of calcium silicate hydrate (C-S-H). This is the same process that takes place during hydration of ordinary Portland cement forming C-S-H. The Ca ions are a product of the early cement reactions.



The difference is that in the case of cement, the C-S-H will precipitate on the cement grain surface, while with water glass the C-S-H will precipitate and gelate the pore water. This will produce the stiffening effect. The end product will be similar to that of cement paste without water glass. The differences include: the amount of calcium hydroxide will be somewhat lower, the amount of C-S-H higher and the amount of alkalis in the pore solutions higher.

2.2.2 Aluminum salt set accelerators

There are two types of aluminum set accelerator: alkali aluminum and aluminum sulphate salts. Both act in a similar way reacting with components in the pore solution forming ettringite. The function is based on the fact that the ettringite crystals form a network of needle-like crystals that stiffen the concrete. The composition of the ettringite mineral is:



The aluminate (Al_2O_3) can also come from other components. Ettringite needs sulphate ions

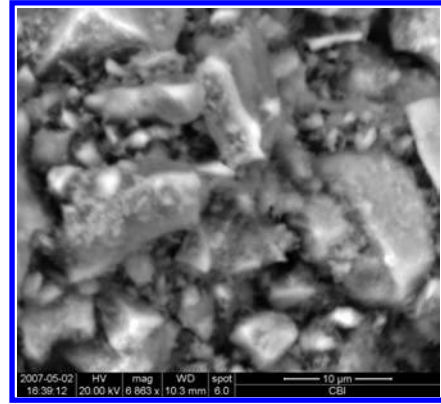


Figure 3. SEM photo taken 1 hour after mixing with alkali free set accelerator. It shows cement grains and a matrix of ettringite needles. The photo is taken in a vacuum on a broken surface. In a wet shotcrete the ettringite crystals would be evenly distributed.

in its structure. The major difference between the two aluminum salts is that the alkali accelerator consumes sulphate and releases alkali ions to the pore solution while the alkali-free accelerator only consumes calcium and hydroxide ions.

Aluminum salts that can be used as set accelerators include:

- Sodium aluminium salt $\text{Na}[\text{Al}(\text{OH})_4]$
- Aluminum sulphate salt $\text{Al}_2(\text{SO}_4)_3$
- Amorphous aluminum hydroxide $\text{Al}(\text{OH})_3$
- Aluminum chloride AlCl_3

In earlier times an alkali accelerator was more common while the alkali-free type dominates today's market. The difference between them is that the alkali-free accelerators do not contain alkalis. Instead they contain an aluminum sulphate salt. The major advantage of the alkali-free accelerator compared to water glass is that the shotcrete becomes stiffer faster and thus it is possible to shoot thicker layers, Figure 3.

3 INVESTIGATION/TEST SERIES

Wet shotcrete is basically ordinary concrete that is mixed with a set accelerator. Most tests are developed to investigate ordinary concrete. Commonly used shrinkage tests measure length changes of cast prisms. Shotcrete, however, can not be cast to prisms thus the major problem is the experimental set up. Shotcrete has to be produced and the length change has to be measured. Moreover, the effect of the set accelerator on the cement hydration must be examined.

To produce a realistic result the concrete must be mixed with the set accelerator in the nozzle. Hand mixing gives an inhomogeneous product. To get a proper result the shotcrete has to be measured as early as possible after mixing. The autogenous shrinkage is normally measured in flexible tubes filled with fluid concrete but it is impossible to get stiff shotcrete into these tubes. Thus fresh shotcrete has to be cut into prisms that are measured. Tests and measurements have been done on cement paste, mortar and concrete. The complete investigation is published in Swedish (Lagerblad et al. 2006, 2007). In this article the results from the cement paste experiments in the above mentioned publications are presented.

The effect of the accelerator on the exothermic hydration reaction has been studied in an isothermal calorimeter. In this instrument temperature is kept constant (20°C) and the amount of energy needed to keep the temperature constant is measured. In the experiments a cement paste is mixed with a set accelerator and immediately put into a cup in the instrument.

Two types of experiments have been performed with pure cement paste. The autogenous shrinkage is measured on cement paste put in elastic rubber containers (condoms) that are put in a water bath. Weight changes are measured. The shrinkage will result in a change in buoyancy leading to a higher weight (as the container is in water). The change in weight can be used to calculate volume change. The method is described in Justnes et al. (1999).

To determine the free shrinkage, cement paste was mixed with the set accelerator and shot into a 250 mm steel mould. The cement paste contained 1 kg of cement, 50 grams of silica fume and 400 grams of water. This gave a w/c ratio of 0.4 and a w/b of 0.38. Gauge studs were installed in the steel mould. Paint spraying equipment driven by compressed air was used. The set accelerator was introduced in the air stream. After spraying steel plates were pushed into the stiff, but still plastic, grout. The steel mould was placed in a humid room at 20°C and was dismantled the next day, resulting in three 250 × 250 × 25 mm³ prisms with gauge studs. The prisms were kept in the humid room for 7 days and then put in a climate chamber with 65% RH.

The mortar was sprayed onto wooden surfaces in the laboratory. It was sprayed with a modified putty sprayer. The shotcrete contained 1540 kg (0–4 mm) aggregate, 500 kg cement, 25 kg silica fume, 236 kg water and 1.5 kg polycarboxylate superplasticizer per m³ concrete. An alkali-free accelerator was introduced in the air stream during spraying. The cake of stiffened grout was cut into prisms and measurement devices were put on them. The first measurement was done within an hour. The prisms were put in a humid room and kept

there for a week. After this first week the prisms were put in a climate chamber with 65% RH.

Shotcrete was also studied in field experiments. It was sprayed onto wooden panel forms. From these panels prisms were cut and gauge studs were glued either on the sides or, as in the normal procedure, to the short surfaces.

In all the experiments fairly coarse grained and slow reacting low alkali sulphate resistant cement (CEM I MH, LA, SR) was used typical of underground shotcrete in Sweden. This type of cement must be used due to relevant Swedish standards. In Sweden it is also common to use mixtures with 5% silica fume (by weight of cement).

4 RESULTS

4.1 *Effect of set accelerators on cement hydration*

The paste was hand mixed and immediately put in the calorimeter. The results show that one of the alkali free set accelerators (Af-1) led to a distinct retardation while the other (Af-2), like water glass, does not effect the onset of the acceleration period (Figure 4). A third type (Af3) leads to a slight retardation.

The differences in effect on retardation are related to the amount added and are thus a result of energy release from the set accelerator reactions. The result show that there is a difference between different brands of alkali free set accelerators where some can lead to distinct retardation. This shows that different alkali free accelerators lead to different effects presumably due to different formulations. Water glass, on the other hand, produces a slight acceleration even at high dosages. The conclusions from this investigation are that there are variations in the composition of alkali-free set accelerators and that some of them lead to retardation with this type of cement. The water glass seems to accelerate the reactions.

A difference can also be found in relation to strength where the reference mix and water glass give a distinctly higher strength after 1 day compared to the mixes with alkali-free accelerator, Figure 5. The 28 day strength is lower than that of the reference paste. Water glass leads to less retardation and thus confirms the results from the calorimeter. These tests were performed at 20°C and one can presume that with lower temperature the retardation will be stronger.

4.2 *Shrinkage of cement paste*

4.2.1 *Autogenous shrinkage*

Autogenous shrinkage is a result of chemical shrinkage within the hydrating cement paste. The

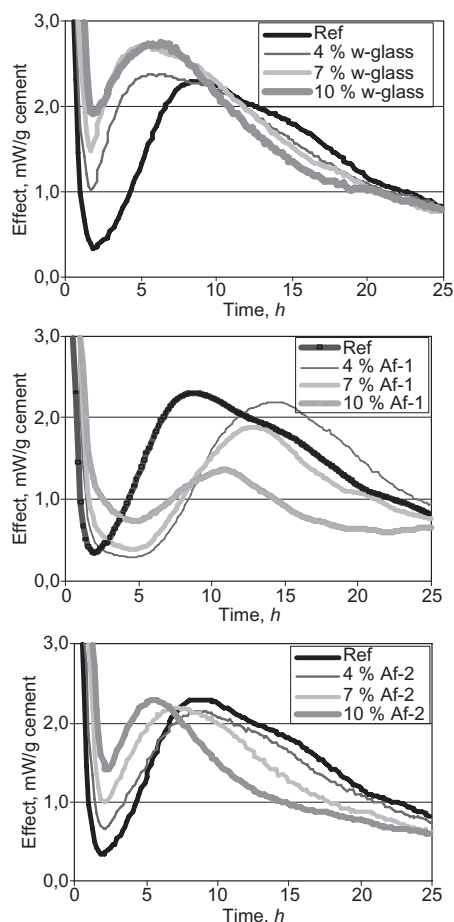


Figure 4. Results from isothermal calorimetry. Two different types of alkali free (Af) and one type of waterglass (Wg) at different dosages.

experiments have been done with cement paste. Two of the pastes were mixed with an alkali-free set accelerator and one was ordinary cement paste. The ones with accelerator were put directly in the elastic rubber after mixing. The results show shrinkage during the first 10 hours and then a flattening of the curves, Figure 6. This was because after this initial time the strength was high enough to withstand the shrinkage. The “shotcrete paste” shrinks somewhat less than the ordinary paste but it was stiff almost from the start. This shows that the set accelerator does not hinder the early chemical shrinkage. In the shotcrete this will result in an increased porosity due to the stiff structure. In the ordinary fluid fresh concrete it will result in setting.

For the “shotcrete”, the results indicate that the autogenous shrinkage does not stop until the real cement hydration and strength start to develop.

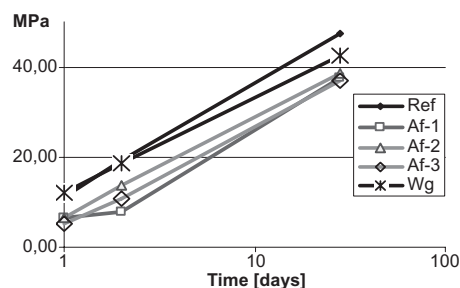


Figure 5. Compressive strength for 1, 2 and 28 days for sprayed cement paste. This was measured on prisms ($40 \times 40 \times 160$ mm). Af = alkali free accelerator, Wg = Water glass.

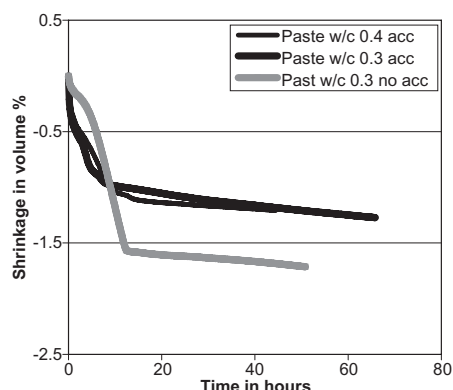


Figure 6. Results from experiments with autogenous shrinkage. The method is described in Section 3.

These results are confirmed by a set of field experiments performed in a tunnel north of Stockholm. In these experiments, described in Azis (2005), real shotcrete as used in the tunnel, was sprayed onto wood panels. Directly after shooting, gauge studs were fixed onto the concrete and they were covered with plastic sheets to prevent drying. This was basically the same procedure used to study autogenous shrinkage. During the first 24 hours, the concrete shrunk around 0.5 %. This shows that for shotcrete it is not sufficient simply to provide a covering or membrane curing, water must also be added.

4.2.2 Paste prisms

Three different types of commercial alkali free set accelerator and one type of water glass were tested. Different dosages were also used. The paste consisted of cement mixed with 5% silica fume and had a water/binder ratio of 0.38. Mixes with 5% silica fume are standard practice in Sweden.

The moulds were dismantled after 20 hours and the prisms were kept in humid conditions for 7 days.

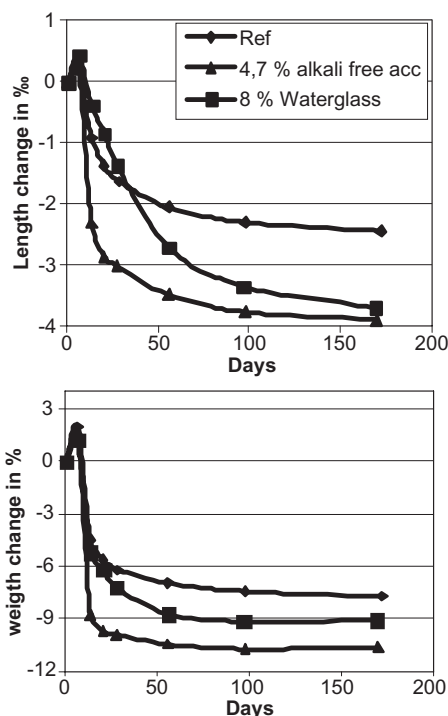


Figure 7. Length and weight change due to drying shrinkage. Results from experiments with cement paste and different types of set accelerators.

Thereafter they were put in a conditioning room at 65% RH. The results show that the prisms during the first week, when kept in humidity, swelled by around 0.3‰. In another experiment in which the specimens were kept in the humid room for 2 weeks they swelled by 0.4‰ (Lagerblad et al. 2007).

When the prisms were put in the conditioning room (65% RH), they started to shrink rapidly. The results show that both the hardened pastes with set accelerators shrank more than the reference concrete. The samples with alkali-free set accelerator shrank most, around 40% more than the reference paste prisms. Moreover the prisms with alkali-free set accelerator lost more water and lost it faster than the prisms with water glass and the reference. Results are presented in Figure 7. These results have been verified with more experiments where it also was found that shrinkage reducing agents reduced the shrinkage (Lagerblad et al. 2007).

5 DISCUSSION

It is a well known phenomenon that concrete shrinks. There are two different mechanisms behind it. Con-

crete, or more precisely the cement paste within concrete, shrinks because the products of cement paste hydration occupy a smaller volume than cement and water. The other mechanism that causes concrete to shrink is loss of water due to evaporation from the cement paste hydrate. The result of the shrinkage depends on the restraint within the concrete itself and on binding to the substrate. In most tests the free shrinkage of the concrete is measured rather than the restrained shrinkage.

The chemical or autogenous shrinkage is 8–9 volume percent when the hydration is completed. The autogenous shrinkage, however, will end when the cement paste is strong enough. Instead the shrinkage will result in an initial internal under pressure and later in an increased porosity.

Experiments show that paste mixed with an alkali free set accelerator after mixing will shrink more than 1 volume percent before the strength is high enough to withstand the shrinkage (Lagerblad et al. 2007). This shows that the set accelerator does not produce enough strength to withstand stresses associated with shrinkage. Normal paste without set accelerator in the same test set up will shrink more than 1.5 volume percent but in this case the fluid paste will set in the form. A 1.0% volume change will lead to a one dimensional shrinkage strain of 0.33%. If we assume 75% aggregate this will lead to a free shrinkage strain of 0.8‰ for the concrete. In an experiment described in Aziz (2005), and also in Lagerblad et al. (2007), panels with real shotcrete were measured following shooting. In this experiment, the panels were covered with plastic to prevent drying. After 24 hours they were water cured. The panels showed during the first day an average shrinkage of around 0.7‰, which is in accordance with the results from the paste experiments on autogenous shrinkage. This shows that water must be added to the fresh shotcrete to avoid shrinkage.

When the real cement hydration has started, the concrete is strong enough to withstand most of the chemical shrinkage but it will shrink when dried. In normal drying shrinkage tests, the prisms are first stored humid for a week and later subjected to drying in a climate chamber. In Sweden the climate chamber normally maintains a RH of 50%. The results from the pastes (Figure 7) show that the drying shrinkage increases from around 2.4 to 3.9‰. The water glass accelerator gives almost the same increase in shrinkage but it is slower. The weight changes show a similar pattern. In a few examples from real shotcrete on wooden panels an increase in shrinkage strain from around 0.75 to 0.9‰ could be observed (Lagerblad et al. 2007). If we assume 75 vol. % aggregate the paste samples would lead to an increase in drying shrinkage strain from 0.60 to 0.98‰, which is somewhat more than in real shotcrete.

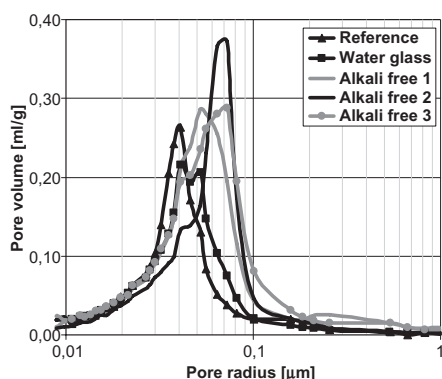


Figure 8. Mercury porosity measured on paste. These are the same pastes that were used in the shrinkage tests (Section 3).

The larger shrinkage and greater loss of water indicate that the shotcrete is more porous than ordinary cast concrete. This can be confirmed by mercury porosity, Figure 8. The results also show that porosity is larger especially in the paste with alkali free set accelerator. This is presumably a result of the fact that hydration occurs in an already stiffened paste.

This data shows that shotcrete is different from ordinary cast concrete. The difference is mainly due to differences in paste structure, which in turn are due to the set accelerator and structure of the paste and concrete. At the wall, the shotcrete has already developed a structure in the case of water-glass based on a loose C-S-H gel, and with the aluminate accelerator on a fibrous network of ettringite crystals. This makes the shotcrete, especially with alkali free set accelerator, very susceptible to early drying and shrinkage. However, the shotcrete also has a different structure in the hardened stage. This must be due to the fact that the main cement hydration reactions take place in an already stiff structure that hinders contraction.

The problem with shrinkage and importance of water curing will be worse in Sweden due to the slow cement. The mechanism will, however, be the same.

6 CONCLUSIONS

Shotcrete is different from ordinary cast concrete. Thus it is not possible to draw conclusions regarding the properties of shotcrete from ordinary cast concrete. The major difference is a result of the fact that the shotcrete is stiffened already at the wall and that major cement reactions take place in a stiff structure. It is, however, not stiff enough to withstand the force of chemical shrinkage. Water

must be added to the shotcrete to avoid shrinkage cracking during the early period. Chemical shrinkage in a stiff but not hard structure also gives the shotcrete more and coarser porosity. This will increase drying shrinkage.

Thus it seems that the problems of cracking in shotcrete are due to the fact that shotcrete shrinks more than ordinary concrete. It also appears that shotcrete with an alkali-free set accelerator develops more shrinkage than the type of water glass accelerator used earlier. This is probably due to the fact that an alkali-free accelerator generates a stiffer structure more quickly than water glass set accelerators.

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REFERENCES

- Aziz, S. 2005. Experimentell undersökning av plastfiber-
armerad sprutbetong. TRITA-BKN. Examensarbete
227, KTH, Betongbyggnad.
- Dove, P.M. & Rimstedt, J.D. 1994. Silica-water
interaction, in: Reviews in mineralogy, Silica, physical
behavior, geochemistry, and material applications, Ed.
Heany, P.J., Prewitt, Gibbs, G.V., Mineralogical Soci-
ety of America, Reviews in Mineralogy. Vol 29, pp.
259–301.
- Hammer, T.A. 1999. Test methods for linear measure-
ments of autogenous shrinkage before setting. In
Tazawa, E. (ed.), *Autogenous shrinkage of
Concrete*, Proceedings of the International Work-
shop organized by JCI, Hiroshima 1999.
- Holt, E.E. & Leivo, M.T. 1999. Autogenous shrinkage
at very early ages. In Tazawa, E. (ed.), *Autogene-
ous shrinkage of Concrete*, Proceedings of the
International Workshop organized by JCI, Hiroshima
1999.
- Justnes, H., Sellevold, E.J., Reyniers, B., Van Loo, D., Van
Gemert, A., Verbowen, F. & Van Gemert, D., 1999. The
influence of cement characteristics on chemical shrink-
age. In Tazawa, E. (ed.), *Autogenous shrinkage
of Concrete*, Proceedings of the International Work-
shop organized by JCI, Hiroshima 1999.
- Lagerblad B., Holmgren J., Fjällberg L. & Vogt C.
2006. Hydratation och krympning hos sprutbetong,
SveBeFo Rapport K24, Stockholm.
- Lagerblad B., Fjällberg L. & Westerholm M. 2007. Sprut-
betongs krympning- modifiering av betongsamman-
sättning, SveBeFo Rapport 86, Stockholm.
- Taylor H.F.M. 1997. Cement Chemistry, 2nd edition,
Thomas Telford Publishing, London.

The use of geotechnical photogrammetry in underground mine development

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ABSTRACT: Fibre Reinforced Shotcrete (FRS) is an integral part of the ground reinforcement and support scheme at Newcrest Mining Limited's Ridgeway Deeps Block Cave gold mine. Resources are implemented into ensuring adequate FRS thickness and coverage. One such resource is photogrammetry. Utilising ADAM Technology's 3DM Analyst Mine Mapping Suite, the geotechnical team is able to capture images of development headings pre- and post-FRS application. These images are processed into Digital Terrain Models, allowing FRS thickness and coverage to be calculated and displayed. Recent trials involving laser scanners and depth indicators for calculating FRS thickness were compared to photogrammetry with promising results and accuracy. Photogrammetry is also used as a geotechnical data collection tool to capture information on the development of headings, and to record geological and geotechnical parameters of the rock-mass.

This chapter outlines the methodology and outcomes of using photogrammetry to determine thickness and coverage of FRS and discusses the value of photogrammetry in comparison to other FRS thickness analysis methods. The use of photogrammetry as a tool in geotechnical mapping is also discussed.

1 INTRODUCTION

1.1 Utilisation

Fibre Reinforced Shotcrete (FRS) is an integral component of Newcrest Mining Limited's Ridgeway gold mine ground support scheme. Ridgeway gold mine is located in the Cadia Valley approximately 30 km south of Orange in NSW, Australia. Mining operations at Ridgeway mine include a Sub Level Cave (SLC), and development of the Ridgeway Deeps Block Cave (BC) operation beneath the SLC. Development of level headings and infrastructure chambers within the SLC and BC operation are via conventional drilling methods using Atlas Copco "Jumbo" drills, and blast methods utilizing Emulsion (Ammonium Nitrate). "Jumbos" are also used to install ground support and reinforcement. FRS is applied by Stratacrete Pty Ltd utilizing Jacon Maxijet rigs.

1.2 Existing FRS quality analysis and control measures

The final thickness of FRS present at any point in a heading is governed by several factors that affect its application. These include over-break of profile, roughness of the excavated surface, overspray,

fall-out and the nozzleman's application method. These factors combined make determining the true thickness of FRS a complex issue.

Adam Technology's 3DM Analyst Mine Mapping Suite comprising *Calibcam*, *Analyst* and *Compare* software are being utilized throughout the development of Ridgeway Deeps BC mine. Methods used for determining the minimum thickness of FRS, as required by Ridgeway's Ground Support System specifications, include the visual inspection of recently mined development cuts, FRS depth testing using penetrometer and borehole analysis, and the use of Adamtech 3DM Photogrammetry systems. A commonly used method of observation involves the use of a stamp on the end of the boom arm of the shotcrete rig; this is pressed into wet FRS to indicate depth. Such point data does not fully represent the full profile thickness but can be a very useful visual QA/QC tool.

Photogrammetry operations and laser scanning trials can be employed to better represent the profile thickness of FRS as discussed below. Newcrest Mining Limited is currently engaged in a FRS improvement research and development project, aiming to improve the design, quality and cost control of FRS application methods.

2 INVESTIGATIONAL METHODS USED TO DETERMINE FRS THICKNESS AND COVERAGE

2.1 Photogrammetry

Photogrammetric methods have been employed to quantify FRS thickness and coverage during mine development. Photogrammetry involves capturing photographs of a mine heading in an arranged order. Using photogrammetry the comparison of images from mechanically scaled headings, with the same heading once freshly sprayed, enables a thickness assessment of FRS to be undertaken. In order to best assess FRS thickness, image capture should ideally be taken after hydroscaling, but because this often delays FRS operations and is rarely carried out, capture is typically taken after only mechanical scaling.

2.2 Equipment set up, and processing

Field equipment and set up includes use of a Canon EOS 5D 12.8 megapixel digital single lens reflex (SLR) camera fitted with a 20 mm lens. The camera is mounted upon a customized bracket that allows movement through 70° horizontally and 50° vertically; this is then mounted onto a tripod fitted with a tribrach to allow leveling. Lighting units are required to provide constant and even light to improve image resolution, as the photographs are taken using 20 to 30 second exposure times. Three high omittance LED battery powered lighting units provide a suitable level of flood lighting.

Adam Technology's 3DM Analyst Mine Mapping Suite is utilized for processing all photogrammetric images. Typically 16 photographs are taken for each development cut, four at each camera station. Using 3DM Calibcam software each image is then georeferenced to mine grid co-ordinates and a Digital Terrain Model (DTM) of the heading produced. The DTM is opened in 3DM Analyst and displays a mesh of up to two million points of the surface of the drive. A textured overlay is draped upon the mesh to show the surface of the heading.

Measurement of FRS thickness and coverage is undertaken using 3DM Analyst DTM Compare software. FRS thickness parameters are input to determine the minimum, mean and maximum FRS thickness at any point in the heading. Results of analysis can be displayed (in accordance with the user's preferences) using full colour thematic mapping capability.

For reproduction purposes of this publication all images have been mapped in grayscale. Figure 1 shows the DTM of a left hand wall illustrating

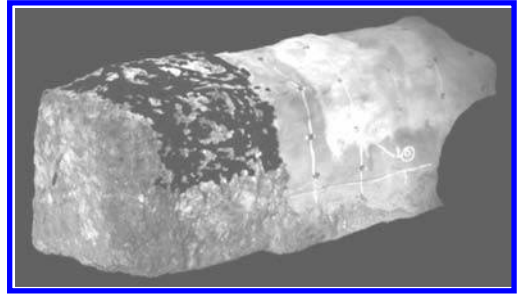


Figure 1. Left Hand Wall with FRS thickness overlay.

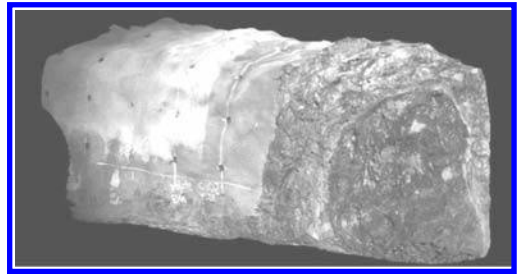


Figure 2. Right hand wall exhibiting photographic overlay.

thematic distribution of FRS thickness. Figure 2 shows the right hand wall of the same drive with photographic overlay. Initial benchmarking of the FRS thickness measurements using 3DM show thickness values to be accurate to approximately ± 5 mm. FRS thickness mapping data taken from drill holes has been cross-referenced confirming the accuracy of 3DM derived FRS thickness.

2.3 Laser scanning trials

Recent trials involving laser scanners were undertaken to provide FRS thickness data for comparison with FRS thickness values derived from photogrammetry. A Riegl 420i laser scanner and Faro LS880 laser scanner were supplied and manned by external surveyors.

The Riegl 420i laser scanner (Figure 3) has a measurement rate of up to 11,000 points per second facilitating the high speed collection of high resolution 3D measurements to an accuracy of ± 4 mm (Loncaric et al, 2009). Scan times per heading were approximately 8 minutes following which data were processed using Riegl RiScan Pro software to produce a DTM. Figure 4 shows a laser scanned DTM prior to FRS application.

Similarly, the Faro LS880 laser scanner is able to capture a 9 mega pixel scan in less than



Figure 3. Riegl 420i laser scanner.

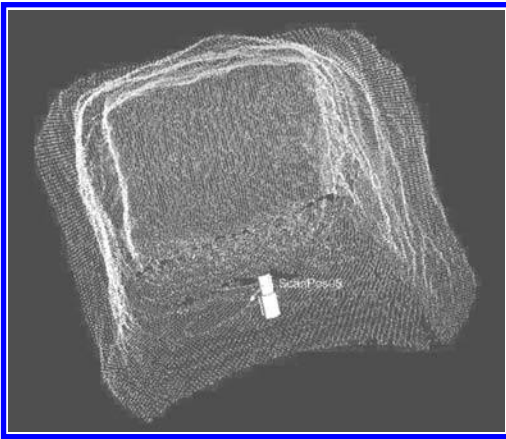


Figure 4. DTM of heading prior to FRS, Riegl laser scanner.

one minute, to an accuracy of 3 mm over 100 m. The Faro scanner was set up in the same position as the Riegl scanner and used the same targets for reference throughout the trial.

The methodology for determining FRS thickness from laser scan data is as before by comparison of pre- and post-FRS DTM data. Figure 5 shows a processed image of a drive pre-FRS. Missing areas in the DTM show where laser scanning failed to capture data, from (Loncaric et al, 2009). Set up was replicated in the drive and the same control point targets used to capture a scan post-FRS of the heading.

Both scanners produced survey accurate images which were suitable for accurate FRS thickness measurements. However, in their current form all the technologies require significant post processing by experienced and trained engineers. None of the technologies provided real time FRS thickness at the face (Loncaric et al, 2009).

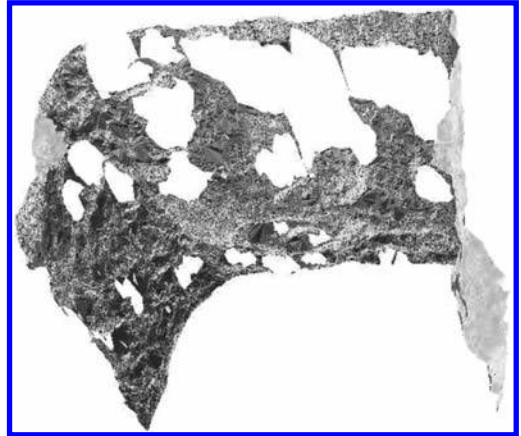


Figure 5. Pre-FRS image captured by Faro laser scanner. Note missing areas in DTM where scanning failed to capture data. (Hi-tech Metrology, personal communication 2008).

2.4 Depth indicators trials

As a further method of FRS thickness assessment FRS Depth Indicators were also trialed. Depth indicators were installed using the boom of the shotcrete rig, and adhered to the rock with quick set adhesives; the length of the indicator shaft equates to the designed FRS thickness. Indicators consist of a shaft and a sacrificial end cone, designed to detach leaving a round disk visible on the surface of the FRS. Subsequent inspection of the heading made it easy to view the indicator discs and judge applied thickness. To enable sufficient coverage approximately 15 to 20 indicators are required in a heading of 4.7 m wide and 4.6 m high.

During trials the nozzleman was able to build up the thickness of FRS to just below the sacrificial cone. The indicators are designed to withstand firing of advancing cuts and Figure 6 shows depth indicators being installed in a development cut. Figure 7 shows an example of a depth indicator used in 50 mm FRS minimum thickness applications.

2.5 Depth testing

Trials to determine thickness of FRS measured from production drill holes were also carried out. Production drill holes are drilled into the FRS once the heading has been fully supported and drive length reached. FRS thickness was measured in each drill hole using a standard steel tape measure. Other methods trialed included the drilling of holes using a hand-held Hilti drill.



Figure 6. Installation of depth indicator with boom of rig. (Loncaric et al. 2009).

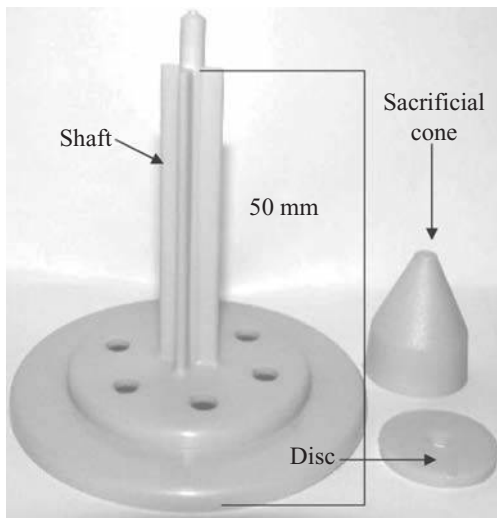


Figure 7. Components of depth indicator. For use in 50 mm minimum FRS thickness ground support design. (Loncaric et al. 2009).

Both these methods are unreliable in determining FRS thickness as the user cannot tell when fresh rock has been intersected as there is often no barrier of hardness between the FRS and the colour of drill powder does not alter.

3 TECHNICAL FINDINGS

3.1 Photogrammetry analysis of FRS thickness

Processing of pre- and post-FRS DTM's allows minimum, mean and maximum FRS thickness to be calculated and displayed, either onto the DTM image directly, or on a wireframe. Thematic mapping enables the user to customize the data displayed

to, for example, show only those areas where FRS thickness is less than 50 mm (Ridgeway Deeps minimum FRS support requirement). Likewise through mapping of mean and maximum thickness values potential areas of 'over-spray' can be displayed.

On an operational level this analysis allows the mine to identify areas where FRS application is either too sparse or too thick, and to understand why this may be occurring. Advice can then be given to the nozzleman about application techniques based upon the analysis.

Utilising the system software, cross-sections and contours can also be generated for any section of both pre- and post-FRS DTM's; exported as DXF files this data can be further analysed using any computer aided design (CAD) software.

3.2 Drill hole FRS depth tests

The testing of FRS thickness from drill holes has proved to be inconclusive. Drill holes in completed development headings present an opportunity to manually measure FRS thickness, provided the drill hole is of sufficient diameter to insert a steel tape measure. However, in order to offer any degree of representative data the number of drill holes required per heading would be unfeasible.

Findings of drill hole direct measurement trials at Ridgeway Deeps showed that typically headings were receiving thicker than necessary FRS application, with only minor areas where application was less than required. Areas that were below the Ridgeway Deeps minimum FRS support requirement were re-sprayed to ensure compliance.

3.3 Laser scanning

Trials involving laser scanners proved relatively successful with the hardware and technology available offering a good means by which to assess FRS thickness. Advantages of using laser scanners include moderate equipment costs (the Faro and Riegl lasers used during trials typically cost between A\$130,000 to A\$200,000), fast on site set-up time (typically 25 minutes), and rapid acquisition of high quality data. Profile scans were equivalent to survey accuracy.

The Faro scanner also had the advantage of being able to provide scan data which is of value for further geological and geotechnical work. The disadvantages of laser scanning were that the techniques do not routinely produce photographic images for reference, and that where images could be recorded (using the Faro LS 880 laser scanner) often the images were of lower resolution and lesser quality than those obtained from photogrammetry.

Data processing was also time consuming and user intensive; FRS thickness being determined by

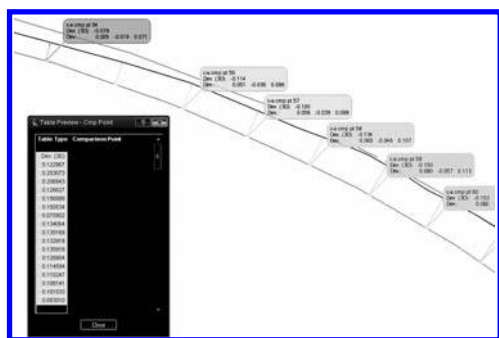


Figure 8. Faro laser scanner FRS thickness calculation. (Hi-tech Metrology, personal communication, 2008).

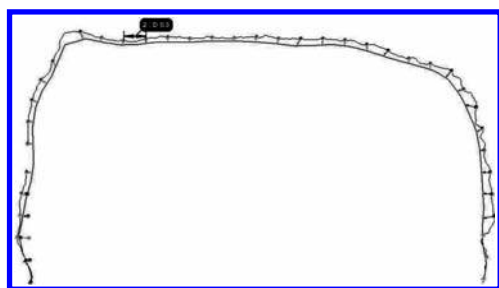


Figure 9. Faro laser scanner. Pre and post FRS profiles. (Hi-tech Metrology, personal communication, 2008).

slicing a cross-section of the data and calculating the difference between the two pre- and post-FRS profiles. Figures 8 and 9 exhibit profile FRS thickness calculations.

4 PHOTOGRAMMETRY IN GEOTECHNICAL ENGINEERING

4.1 Use of photogrammetry in geotechnical mapping

Photogrammetry is an extremely powerful observation tool for geotechnical, geological and mine engineering QA/QC. 3DM Analyst is used to collect geological and geotechnical rock-mass data allowing lithology and rock structure to be viewed at a later date and mapped accordingly. Capturing a heading profile prior to FRS application allows the user to map joint sets, lithology, over-break/deformation, and to measure distances between any two points on the profile.

For mapping purposes the software is utilised for identifying structures and joint sets by simply selecting along the structure to create a plane “halo” which will display accurate dip and dip

direction of the selected structure. This “halo” can then be exported as a DXF and imported into many CAD and plotting packages. This is extremely useful data for determining the extent of large scale faults which can be used as drive scale control for creating wireframes of local and regional scale structures. Figure 10 shows a fault at Ridgeway Deeps that intersects a mining level, this image was produced using photogrammetry data from several development drives.

The DTM wireframes generated by the Analyst software can also be exported as a point file (.pts) into any CAD software and used in an ‘actual as built’ over ‘initial design’ comparison. Where areas within the profile are not excavated to ‘as built’ design, these ‘tight’ areas of profile can be identified and removed by means of mechanical scaling. As a QA/QC tool the system also proves to be very effective in allowing the user to check the ground reinforcement scheme installation against design and the viewing of stamps for FRS thickness checks.

4.2 3DM scan-line mapping trial

Scan-line mapping using photogrammetry was trialed on a mining level which contained a distribution of lithologies. The purpose of the trial was to compare spatial location of rock-mass discontinuities, trace length and discontinuity orientation data derived from conventional scan-line mapping to discontinuity data obtained from photogrammetry images.

Several perceived benefits of photogrammetry over conventional scan-line mapping include the ability to map the rock-mass for the entire development drive, improved estimation of discontinuity orientation and geometry, and increased sampling speed, leading to a wider sampling of the rock-mass and safety improvements due to lessening exposure of personnel to unscreened ground.

The trial relied on the direct comparison of mapping results for the same location. One engineer manually scan-line mapped a heading, while another analyzed and digitized the images of the same heading. A total scan-line length of 27.85 m

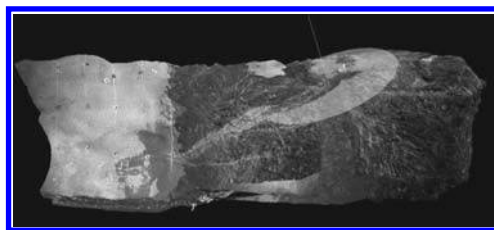


Figure 10. DTM of heading intersected by regional fault. Note plane “halo” overlain on fault.

was analysed; 116 discontinuities were sampled via conventional mapping methods compared to 112 discontinuities sampled using 3DM methods.

The drive was captured with the standard photogrammetry set up and images digitised into DTMs using Adamtech's Compare software. A line was drawn upon the DTM to signify a scan-line. All cross cutting discontinuities were captured using the plane tool. Orientation, trace length and fracture frequency (FF/m) data were all captured. Vertical scan-lines were also captured equi-distant along the horizontal. Figure 11 illustrates a typical scan-line.

The spatial distribution of discontinuities found using both conventional and 3DM scan-line techniques are shown in Figure 12. It is apparent that there is a disparity of trace lengths. The location of major discontinuities, greater than 1 m in length, recorded using conventional mapping are generally not identified in the 3DM images, i.e. discontinuities measured using 3DM had significantly smaller trace lengths compared to those recorded from conventional mapping. Possible explanations for this include the orientation of the camera to the scan-line; the images were captured oblique to the heading, and many of the images once processed were blurry under high magnification (Figure 13). Sections of the image were also blurred and distorted by areas of wall protruding from the profile.

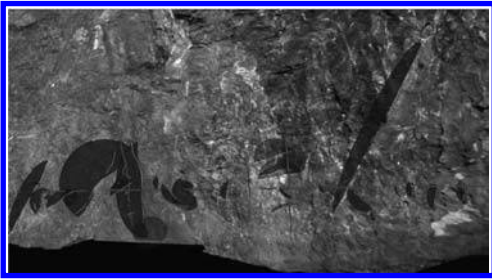


Figure 11. DTM of Left hand wall. Scan-line is drawn along wall with all joints exhibited with planes. Orientations are exported to stereonet.

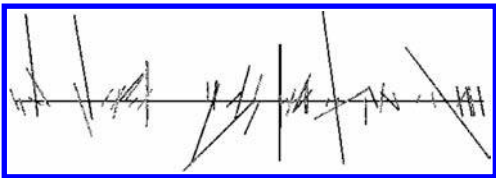


Figure 12. Location and trace length of discontinuities along a scan-line. Longer trace lengths represent conventional mapping, while the short traces represent discontinuities captured by 3DM software.

Identification of different rock types, areas of alteration and areas of blast damage were also difficult to distinguish from photogrammetric images, as were discontinuities striking sub-perpendicular to the profile. A second trial capturing images with the camera set up perpendicular to the scan-line was undertaken. This involved photographing the heading wall and backs from stations approximately 1.5 m apart along a 15 m length of drive. This spacing allowed a 60% overlap of images (captured with a 20 mm fixed lens on a Canon 5D camera body). Data capture of both drive walls took around 25 minutes, with a further 25 minutes required for processing. The image quality was vastly improved on the previous trial methods. Figures 14 and 15 show camera positions used for data capture in the perpendicular trial.

4.3 Capturing over-break in mined profiles

Photogrammetry data also proved to be useful for profile over-break studies i.e. the identification of areas where commonly less competent areas of rock

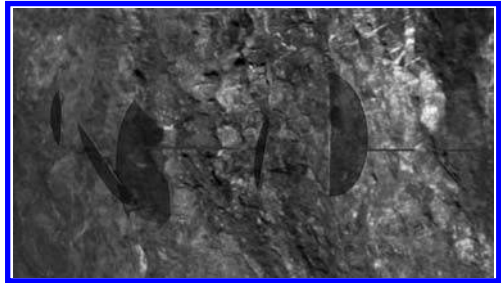


Figure 13. Example of a low resolution photo captured with conventional 3DM.

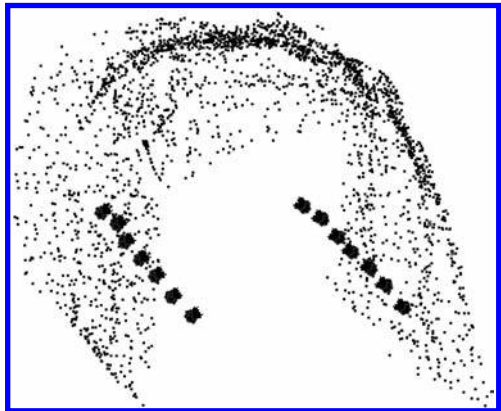


Figure 14. DTM displayed as point data. Camera positions used for data capture during perpendicular trial.

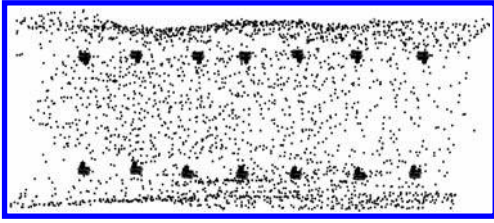


Figure 15. Camera positions in plan view. DTM displayed as point data.



Figure 16. Photographic drape of combined wall images overlain on DTM.

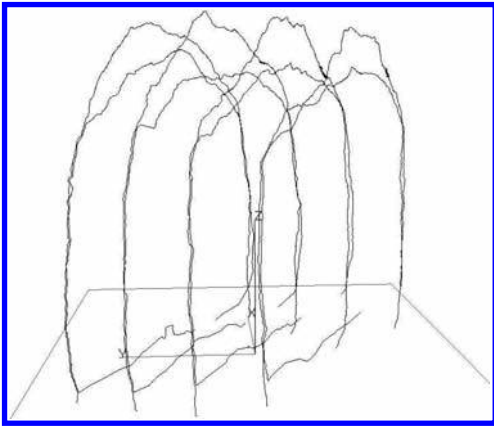


Figure 17. Cross section of heading of approximately 5 m length. Sections of smaller profile are pre-scaling. Sections overlain with increased profile height are post-scaling.

are subject to over-scaling. Figures 17, 18 and 19, produced from photogrammetric data, include cross sections of several development headings exhibiting considerable increases to overall profile shape and size after mechanical scaling.

4.4 Damage mapping in production drives

The system software has the ability to compare, and display disparities between two DTMs, a function which, given regular data input can be used to

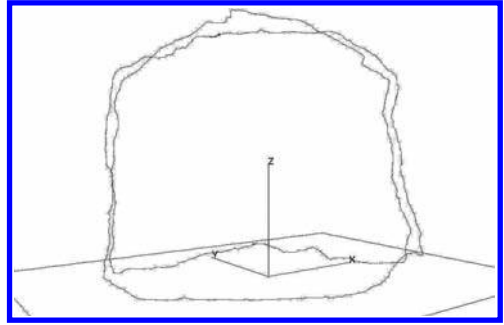


Figure 18. Cross section of heading exhibiting increase in profile after mechanical scaling.

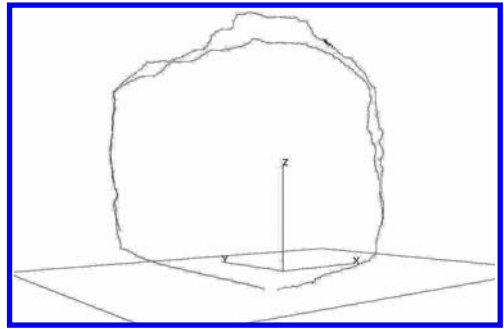


Figure 19. Cross section of heading exhibiting increase in profile after mechanical scaling.

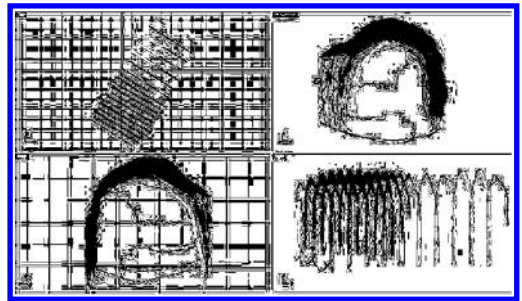


Figure 20. Multi displays of drive profile. Shaded area denotes width difference of profile.

effectively map and record damage to underground workings.

Once processed using Calibcam and Analyst software, data from the same drive heading can be interrogated to determine levels of drive deformation or convergence. Drive profile comparison can be undertaken using a CAD package such as Surpac or Rhino; however significant post processing is required. Figure 20 shows such a comparison

using a CAD package to calculate the difference in drive width pre and post deformation.

4.5 Recording drive convergence

Drive profile convergence at Ridgeway Deeps is measured regularly using a standard digital tape extensometer. Threaded pins in conjunction with Resistance Wire Extensometers (RWE) form the basis of instrumentation used within the Block Cave Undercut level at Ridgeway Deeps. Data is recorded as a reduction in drive width, and can provide an understanding of the depth of failure around a drive excavation.

Designated areas of drive have threaded pins 200 mm long installed into the rock walls to which a digital tape extensometer can be attached. Benefits of this system include ease of set up and fast data collection time, each reading taking less than 2 minutes to capture. However the convergence measurements taken with this method do not allow the engineer to calculate the direction of movement.

To establish the direction of convergence a trial was undertaken using photogrammetry with the perpendicular camera set up. The selected drive had threaded pins installed to provide reference points during trial measurements. Approximately 10 m length of drive was captured. This trial required careful planning as the installation of ventilation ducting (vent bag) caused the 3DM Calibcam software to reject the images prior to digitization as minor amounts of movement was recorded in the vent bag. The vent bag was closed, but minor amounts of vibration were still noticed. Subsequently only headings that had sufficient distance from the face to vent bag could be recorded (as removal of vent bag would impinge on mining development). Figures 21, 22 and 23 show a drive being marked up ready for the 3DM convergence capture.

Digital tape extensometer readings were taken, and the width between control points also measured, using a Leica Disto A5 and Analyst line tool. Four control points were established on the drive walls. The same locations, camera height and distance from walls were methodically replicated for each capture to ensure tight tolerances in data capture. Table 1 summarises the measurements taken allowing a comparison of data from the digital tape extensometer, Leica Disto A5 and Analyst measure line tool. Comparisons of accuracy of all methods were equivalent to tenths of one millimetre.

Data collected showed an average drive convergence of 50.54 mm. The area was then lost to the undercut brow being advanced along the drive.

Photogrammetric images were used to create a thematic map from which minimum and maxi-



Figure 21. Setting up for convergence data capture.



Figure 22. Western wall. Control points 1 and 2.



Figure 23. One of the authors measuring distance between control points 1 and 3.

mum points were identified relative to that of changes established with the line measurements. The thematic overlay showed areas that had received recent deformation. The area if significantly changed can be recorded and a line drawn perpendicular from the point of deformation

to the opposing point in the profile, and width recorded. Figures 24 and 25 illustrate DTMs utilizing the line function to measure drive width change. The vector of damage can be established using this method. Numerical modeling can utilize this data to calibrate deformation orientations within a developing mine.

Table 1. Convergence data. Site 1 Undercut Mining Level.

Method	Control points (m)		Drive width change (mm)	
	1–3	2–4	1–3	2–4
Leica Disto	5.028	4.985	50.40	20.30
Line tool	5.030	4.980	80.00	60.00
Tape exto	5.022	4.983	70.85	60.02
Rock bolt*	5.160	5.100		60.00

* Rock bolt common for all measurements. Located in shoulder of drive.

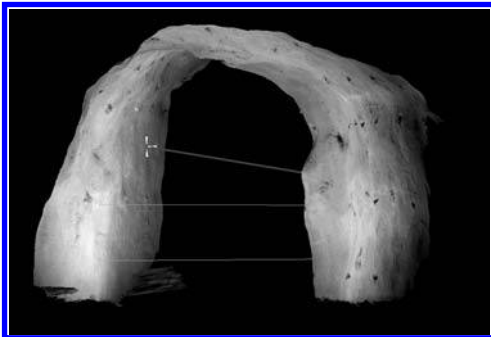


Figure 24. DTM exhibiting line functions to measure distance between control points. Note line tool measuring distance to bolt in left hand shoulder.

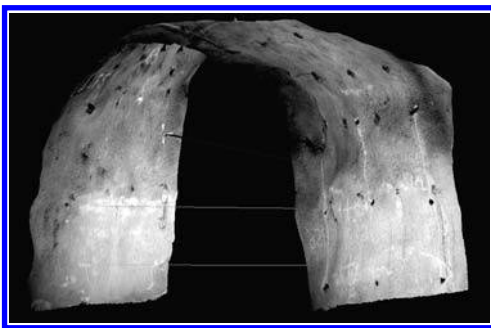


Figure 25. DTM exhibiting line functions to measure distance between control points.

5 CONCLUSIONS

Photogrammetry employed at Ridgeway has allowed the geotechnical team to develop innovative methods of FRS thickness measurement and geotechnical mapping in hard rock mining. As a result of extensive research and development trials the methods of photogrammetry are now utilized as an innovative tool. Photogrammetry provides a cost effective method of capturing, collating and storing high quality data particularly in comparison to equivalent laser scanning technology.

Currently the most efficient method of FRS thickness investigation is the use of the stamp on the end of the shotcrete rig boom. Combined with depth indicators, an efficient and cost effective inspection can be undertaken to audit the FRS thickness. If a more concise understanding of FRS thickness is needed, for example in a crusher chamber, photogrammetry in its current proven form would create a high quality, repeatable data record of the supported profile.

Photogrammetry as a geotechnical mapping tool has enabled a greater understanding of rock-mass properties at Ridgeway Deeps to be gained and it will continue to be used to supplement conventional geotechnical mapping. Further trials involving shotcrete rig-mounted laser scanners and photogrammetry to investigate real time FRS thickness are recommended.

ACKNOWLEDGEMENTS

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REFERENCES

- Birch, J. 2008. "Data Acquisition with 3DM Analyst Mine Mapping Suite". *SHIRMS 2008*, Perth, pp. 531–540.
- Grobler, H.P., Poropat, G. and Guest, A.R. 2003. "Photogrammetry for Structural Mapping in Mining". *ISRM 10th Congress Technology Roadmap for Rock Mechanics*.
- Loncaric, A.J., Loomes, A., Lett, J. and Emmi, J. 2009. "Control and Measurement of Shotcrete Thickness". *First International Seminar on Safe and Rapid Development Mining*. ACG, Perth, pp. 101–110.

Testing of unreinforced masonry walls seismically retrofitted with ECC shotcrete

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ABSTRACT: Engineered Cementitious Composite (ECC) is a type of strain-hardening cement composite that was sprayed onto UnReinforced Masonry (URM) walls in order to determine the suitability of using this method of application as a seismic retrofitting technique for URM buildings. From studies considering the mix design of ECC it was determined that sand is a crucial component influencing the strain-hardening behaviour. Following improvement of the sand used, an ECC with an ultimate tensile strain capacity of 2% was obtained and three brick high prism samples with ECC sprayed onto one face were tested in axial compression and found to exhibit five times the energy absorption of the non-retrofitted prisms when using 10 mm of ECC reinforcement. Retrofitted masonry wallettes were tested to determine their diagonal in-plane response and results showed that ECC had successfully modified the brittle behaviour of non-retrofitted samples, to instead exhibit ductile behaviour. Finally, two 4.1 m high walls were constructed to test out-of-plane response. It was determined that 30 mm of ECC was ineffective when sprayed on the tension surface but increased the load capacity by a factor of 1.6 when applied on the compression surface. It is concluded that ECC is an effective material for in-plane seismic retrofitting of URM buildings but additional reinforcement is required in order to adequately resist the out-of-plane loading.

1 INTRODUCTION

Unreinforced masonry (URM) buildings comprise a significant proportion of New Zealand's heritage building stock, and were approximately built between 1880–1935. With the number of URM buildings estimated at approximately 3600 (Russell & Ingham, 2008), seismic forces were not appropriately considered when these buildings were originally designed. The absence of tensile-resistant structural elements required to sustain earthquake loading, together with the frequent seismic activity that is expected due to New Zealand's geographical location, result in seismic retrofitting of many of these buildings being necessary. The need for seismic retrofitting of URM buildings was further demonstrated by the 2007 Gisborne Earthquake which caused damage to at least 37 URM buildings, causing both in-plane cracks and numerous out-of-plane failures of parapets.

Examples of damage to URM buildings occurring in the 2007 Gisborne Earthquake are shown in Figure 1. The observed damage showed that non-retrofitted URM buildings pose a threat to public safety and that their loss is detrimental to the preservation of New Zealand's architectural heritage. To ensure satisfactory performance in future

earthquakes, appropriate seismic retrofit must be undertaken in a cost-effective manner.

Shotcreting using concrete with mix ratios of one part cement to three part sand has been a conventional technique for retrofitting of URM buildings. However, conventional shotcrete generally requires a minimum thickness of 50 mm to incorporate steel mesh that is embedded within the concrete in order to provide the required tensile strength (Fillitsa & Michael, 1992). The retrofit procedure is complicated as additional connection is required between the conventional shotcrete and the masonry. Formwork is also required to secure the mesh and concrete to the masonry wall (Kahn, 1984).



Figure 1. Out-of-plane failure of parapets and in-plane failure of piers.

Engineered Cementitious Composites (ECC) are cement composites reinforced with synthetic fibres. They are tailored using theories of micro-mechanics and fracture mechanics to achieve strain-hardening through the process of micro-cracking. In order to use ECC as an effective material for seismic retrofitting, emphasis must be placed on the success of producing ECC with a sufficient tensile strain capacity so that it can form tension resisting elements in a seismic retrofit of a URM building. Strain-hardening behaviour can provide up to 5% tensile strain, depending on the ECC application method (Li & Fischer, 2002). For retrofitting purposes, shotcreting is typically chosen as an application method because it is an economical method compared to casting of ECC (Kanda et al., 2003). With shotcreting mix design, additional attention regarding the pumpability of ECC is required, which generally compromises the tensile straining ability of ECC. However, even with these compromises, ECC is still able to demonstrate tensile strains of approximately 2%, making ECC an ideal material for retrofitting of unreinforced masonry buildings.

2 ECC MIX DESIGN

The mix design used in this study was based on previous research performed by Kim et al. (2003), who produced an ECC with a tensile strain capacity of 2%. However, the authors found that using the same design resulted in poor ECC performance and difficulties in production. Observed inconsistencies in the ECC as produced included no strain-hardening behaviour, excess water bleeding, and segregation of materials. Therefore it was deemed necessary to make adjustments to the design in order to compensate for the difference in performance that arose from the use of local New Zealand materials. The materials that affect the tensile performance of ECC were researched, and their influence is summarised in Table 1.

Table 1. Materials used in ECC mix design.

Material	Factors that affect tensile performance
Fibres	Aspect ratio (Length of fibre/Diameter of fibre), Surface coating
Sand	Average Size, Surface roundness
Cement	Cement content
Water	Water content
Fly Ash (FA)	Aggregate size and FA content
Air Entrainment (AER)	AER content

From Table 1 the materials that influence the tensile performance of ECC can be listed as: fibres, sand, cement, fly ash and air entrainer. As the fibres are standardised and sourced from the same manufacturer as used by previous researchers, it was determined that this factor could be eliminated from the investigation. Cement and water were also eliminated due to their standard properties regardless of geographical location. Fly ash is highly variable depending on source location and even within the same source, the variability between each batch can be relatively high. However as the average size of the fly ash particles is comparatively small, at an average of approximately 40 μm , it was determined that this constituent should not contribute to significant differences in performance between the published and local design. Having eliminated all materials except the sand and air entrainer, it was obvious that focus should be placed on the possible influence of these two constituents on inconsistency in the ECC performance.

Sand was prioritised in the investigation as it constitutes a larger proportion of the mix volume. The original sand used in the ECC mix design was dune sand with an average size of 300 μm . At this size, difficulties arise in allowing fibre slippage through the matrix, thus causing strain-softening behaviour. The investigation conducted by Li et al. (2001) suggested that sand particles having a size of 100 μm are more desirable in order to achieve strain-hardening behaviour. Therefore smaller pit sand with an average diameter of 200 μm was sourced and trialed for use in the ECC. However these results were also unsatisfactory with regard to the strain-hardening behaviour. When examining the pit sand using a microscope (see Figure 2), it was identified that the pit sand had rough edges. These rough edges were believed to scratch into the fibres, preventing them from slipping and causing premature fibre rupture under stress, ultimately leading to unsatisfactory strain-softening behaviour.

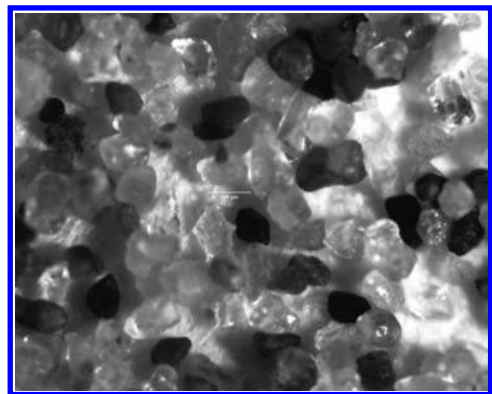


Figure 2. Microscopic view of 200 μm diameter pit sand with 3000 $\times$ magnification with respect to original size.

Having determined that the sand surface texture was crucial to the strain-hardening behaviour, it was necessary to replace the pit sand with silica beach sand. Beach sand has smoother surfaces in comparison with pit sand due to the difference in formation. Pit sand is generally produced as a result of cracking and stress, whereas beach sand is formed from erosion by water and therefore has smoother surfaces. Silica beach sand from Auckland was sourced. The silica sand supplier claimed that the sand had an average grain size of 200 μm , but when viewed under the microscope (see Figure 3), it was determined that the actual grain size was approximately 250 μm . Nevertheless, the silica sand had a much smoother surface when compared with pit sand and was therefore trialed in the ECC mix.

The resulting ECC successfully exhibited strain-hardening behaviour, with an ultimate strain capacity of approximately 1.6%. Even though the silica sand was successful in the production of ECC with strain-hardening behaviour, a further attempt was made to improve the ultimate tensile strain capacity in order to increase ECC's performance as a retrofit material. However, attempts to locate any smaller silica sands from New Zealand were unsuccessful so a silica beach sand was sourced from Perth, Australia. This sand has a similar smooth surface to Auckland beach sand, but is significantly smaller with an average grain size of approximately 150 μm , which was similar to the grain size reported by Li et al. (2001) and was consequently chosen as the final sand to be used in the ECC mix design. The microscopic view of this sand is shown in Figure 4. Note that Figures 2–4 are reproduced at the same scale and can be directly compared in terms of grain size and roughness.

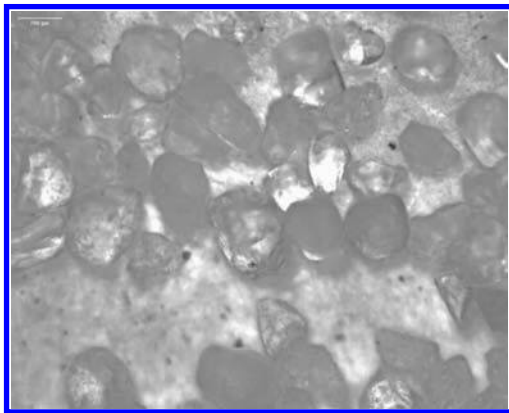


Figure 3. Auckland silica beach sand with 250 μm diameter with 3000 $\times$ magnification.

2.1 Final mix design

The final ECC mix design proportions are shown in Table 2. The mix design was produced and trialed to determine the tensile strength, tensile strain capacity, and compression strength. Compression strength was determined from cylinder compression tests as for normal concrete.

For tensile tests, dog bone shaped samples were cut out of ECC that was shotcreted onto plywood walls. Other means of producing the samples, such as directly spraying the ECC into the moulds, were also trialed. However from Figure 5 it can be seen that only the samples cut directly from the ECC on the walls had well dispersed fibre distribution as shown in the failure cross section. Dog bone samples are shown in Figure 6. The samples were sprayed with water mist then sealed with a wrap until testing.

Due to the specialized nature of equipment required to measure the tensile strength of these comparatively small dog bone samples, the tensile testing was performed by an American laboratory. The ECC materials were mixed using an ordinary Hobart mixer. The determined material properties are listed in Table 3 and a tensile stress-strain graph of ECC shown in Figure 7.

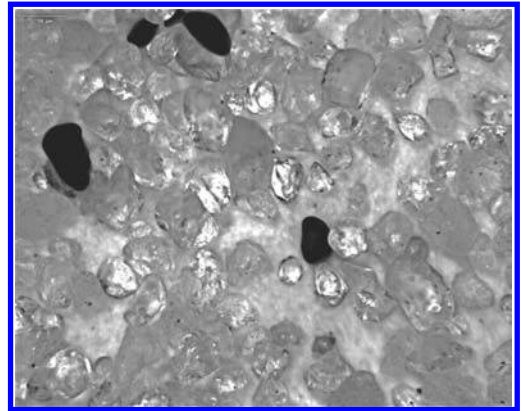


Figure 4. Perth silica beach sand with 150 μm diameter with 3000 $\times$ magnification.

Table 2. Mix ratios of ECC composite.

Materials	Proportions (kg/m ³)
Sand	64
Cement	76
CA Cement	4
Fly Ash	24
Water	37.4
Super Plasticiser	0.26
Stabiliser	0.041
Fibres	2% by Volume

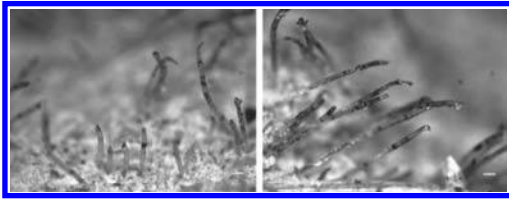


Figure 5. Failure cross sections of samples, a) Cut out of sprayed ECC walls, b) sprayed into molds.



Figure 6. Tensile test dog bone samples.

Table 3. Determined material properties of ECC from testing.

Properties	Values
Compression strength	38.5 MPa
Tensile strength	3.0 MPa
Maximum tensile strain	2.3%
Flexural strength	6.7 MPa

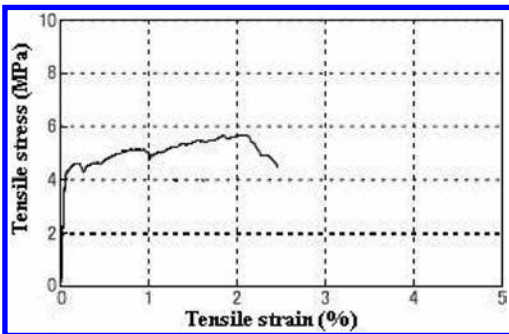


Figure 7. Stress-strain graph of ECC under tensile test.

3 MASONRY PRISM TESTING

Three brick stacked masonry prisms were constructed and tested for two purposes. The first purpose was to test the maximum thickness of ECC shotcrete that could be applied onto a masonry surface in a single placement without debonding due to selfweight. The second purpose for testing prisms was to undertake a comparative study regarding the shear strength capacity of prisms reinforced with ECC, performed by Bruedern et al. (2008) at the Technical University of Dresden, Germany.

The bricks used to build the samples required in this study were recycled from a demolished Irish bar in Auckland, built during the 1930s. The mortar was ASTM type O lime mortar with the ratio between cement, lime and sand being 1:2:9. The material properties can be found in Table 4.

3.1 Testing program

A total of 12 masonry prisms were shotcreted with various ECC thicknesses incrementing from 10 mm up to 30 mm, all applied in one single spray. Three prisms were sprayed for each of the 10 mm, 20 mm and 30 mm ECC thickness, while six additional prisms were left as controls and were not shotcreted. All the masonry sample surfaces were brushed and wetted prior to shotcreting as recommended by Kahn (1984). The shotcreting equipment included an air pump and a wet-mix shotcrete mixer, which were both capable of delivering 0.8 MPa of pressure. A minimum nozzle head diameter at least equal to the length of fibre used is required to prevent fibre clumping. The fibre used in this study had a length of 8 mm and the nozzle head used had a diameter of 16 mm.

From the observation of the shotcreting process it was apparent that in one spray the thickness of ECC ranged between 10–20 mm. Rapid spraying to form an ECC layer thickness of 30 mm was attempted, but delamination occurred, suggesting that it was not feasible to spray more than 20 mm of ECC onto a masonry surface at once. Instead, ECC should be applied onto the masonry surface with an approximate thickness of 10 mm and a 30 minute gap between each layer should be provided so that the previous layer has sufficient time to harden onto the masonry surface.

The prisms with a 30 mm thickness of ECC were re-sprayed with ECC, but this time to a thickness of

Table 4. Compressive strength of brick, mortar, and masonry in MPa.

Brick f'_b	Mortar f'_j	Masonry f'_m
5.7	1.8	4.2

10 mm each time with an interval of 30 minutes. Three prisms from each of the 10, 20 and 30 mm ECC reinforcement were tested for shear strength at 28 days using the test setup shown in Figure 8. The samples were sprayed with water mist until testing date.

3.2 Testing results

Figure 9 shows the results of prism testing. It can be seen that as the reinforcement layer thickness increased, the strength increased proportionally. It can be seen that prisms with 10 and 20 mm of ECC exhibited a similar force-displacement characteristic, with an initial peak load that dropped rapidly as the mortar joints failed, followed by delamination of the ECC layer, as indicated by the fluctuating load approximately 1.5 kN. For the prisms with 30 mm of ECC, a significantly higher amount of energy was obtained when compared with other prisms. The prism testing demonstrated the ability of ECC reinforcement to increase the energy absorption capacity of masonry during an earthquake. However, as a significant proportion of fly ash was used in the mix design, more investigations should be made on the long term property changes of ECC, as the strain capacity of ECC may reduce while the strength increases over time. The possibility of ECC becoming more brittle long term exists and needs to be verified in a future study.

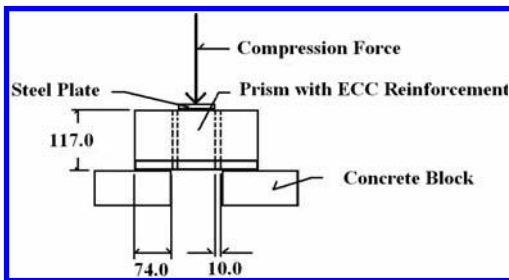


Figure 8. Test setup of prism testing (reproduced from Bruedern et al. (2008)).

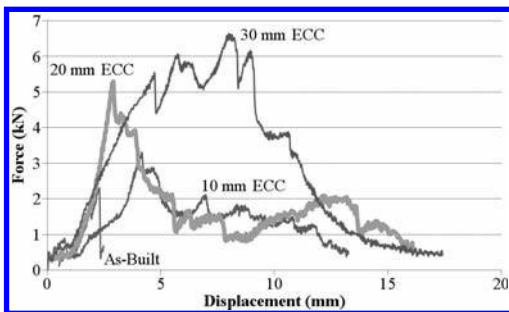


Figure 9. Force-displacement graph of prisms reinforced with 10, 20 and 30 mm ECC.

4 MASONRY IN-PLANE DIAGONAL SHEAR TESTING

Four principal failure modes exist for in-plane failure of unreinforced masonry walls. The failure modes include diagonal cracking, shear sliding, toe crushing and rocking (Magenes & Calvi, 1997). The failure mechanisms of shear sliding and rocking are displacement based and dissipate more energy during earthquakes, whereas the other two failure modes are force based brittle failure modes and are therefore less desirable.

Five masonry wallettes were constructed to test the effect of the ECC shotcrete reinforcement on in-plane diagonal shear strength. The wallettes had approximate dimensions of 1.2 m high by 1.2 m wide, with thickness being 220 mm. Common bond was adopted in construction, with one header course every fourth level. The front view of these wallettes can be seen in Figure 10.

4.1 Testing program

The first wallette was not retrofitted and was left as the control unit, while wallettes two, three and four were shotcreted 14 days after construction with the thickness of ECC being 10, 20 and 30 mm. The fifth wallette was also shotcreted with 20 mm ECC but control pins were placed between the wallette and the ECC layer at approximately every 0.5 m. These pins were placed in the mortar beds but not in locations where the head-joint meet the bed-joint. It was intended that these control pins provide a method

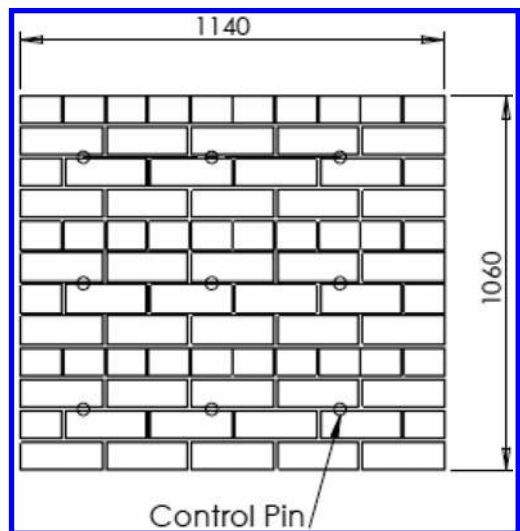


Figure 10. Front view of wallettes constructed showing approximate positions of control pins (pins were only installed on W5).

for the engineer to control the thickness of ECC on site, as well as providing better shear force transfer between the ECC and masonry surface. The test matrix of the wallettes is reported in Table 5.

The ECC shotcrete was applied onto the brick surface to an approximate thickness of 10 mm for each applied layer, and a 30 minute interval was adopted between the applications of each layer to allow the previous layer sufficient time to stiffen up, so that the new layer was sprayed onto a relatively hard surface. It was previously reported that when ECC was applied onto a soft surface the fibres tended to be less well aligned to the plane of the wall and became less effective in strain-hardening. A comparison between shotcreting on a soft surface and on a hard surface is shown in Figure 11.

The ECC shotcrete reinforcement was cured using a constant spray of water mist on the surface until the testing date which was 28 days after applying the ECC shotcrete. Testing was performed using a modified version of the diagonal shear test specified in ASTM E-519-02 (ASTM, 2002). A schematic of the test setup is illustrated in Figure 12.

As can be seen in Figure 12, steel angles were placed at two corners of the wallettes and dental plaster was used to fill between the steel angle surfaces and the wallette surfaces to ensure even

Table 5. Test matrix of wallettes.

Wallet	ECC overlay thickness (mm)	Note
W1	0	Control wall
W2	10	
W3	20	
W4	30	
W5	20	Control pins at 0.5 m spacing

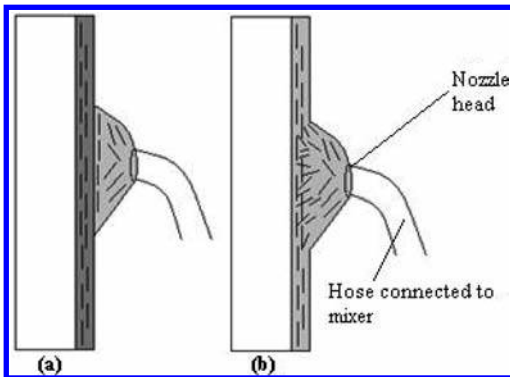


Figure 11. Illustration showing fibre orientation when a) sprayed after previous layer stiffened b) immediately after spraying previous layer.

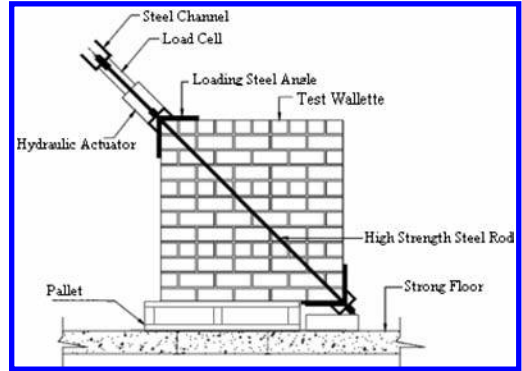


Figure 12. Modified diagonal shear test setup.

stress transmission. High tensile strength steel rods connected the two steel angles together with one rod on each side of the wallette. Hydraulic jacks and load cells were connected to the top steel angles. Forces were incrementally applied using the hydraulic jack. Strain gauges were applied on both surfaces of the wallette to measure the associated displacement of the wall as the load was applied.

4.2 Test results

The shear stress versus shear strain relationship for all five wallettes is shown in Figure 13.

The as-built wall (W1) failed with a major diagonal crack as shown in Figure 14. The diagonal crack formed parallel to the applied force, and the crack path was primarily through the head joints and bed joints, although there were some places where the crack went through the brick itself. Overall the wallette failed as anticipated, with a diagonal crack forming through the joints due to a weak mortar strength compared to the brick strength. The brittle failure exhibited by the as-built wall (Figure 14) confirmed the necessity for retrofitting in order to achieve a more ductile failure mode. W1 achieved a shear stress of 0.07 MPa and a drift of 0.9%.

All wallettes retrofitted with ECC exhibited a ductile failure behaviour, which is more desirable as more energy is dissipated during earthquakes. W2 with only 10 mm of ECC exhibited a significant improvement in performance over the as-built wallette, with the shear strength increasing to 0.25 MPa and the drift increased to 4.7%. During testing (see Figure 15) multiple diagonal cracks were observed in comparison with the single crack that developed in the as-built wallette. The multiple diagonal cracks suggest a better stress distribution within the wall. The formation of multiple cracks is thought to be due to the stress transmission from the brick to the ECC layer. As the crack opens in the brick wallette, the stress at that location is

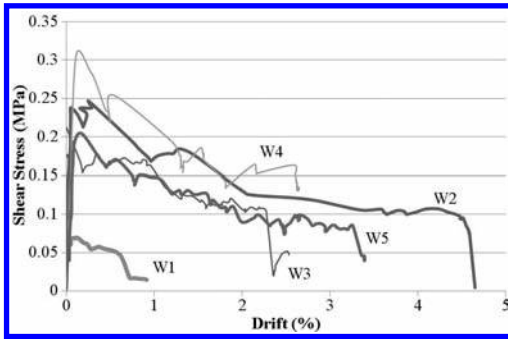


Figure 13. Shear stress and drift graphs for the 5 wallettes.



Figure 14. Major diagonal crack in the as-built wall.



Figure 15. Multiple diagonal crack on ECC reinforced wallette.

immediately supported by the much stronger ECC layer. After this, other cracks open in other weaker parts of the brick wall and the process repeats until the stress in the ECC layer exceeds the capacity of ECC layer. The multiple diagonal cracks mentioned occurred through the bed and head joints as for the as-built wallette.

W3 and W4 exhibited similar failure behaviour modes as observed for W2, except for the

magnitude of increase in maximum shear stress and maximum drift when compared with W1. For W3 the ultimate shear stress was 0.23 MPa, with the drift being 2.5%. W3's decrease in maximum drift when compared with W2 was expected, as a thicker layer of ECC increased the stiffness of the wall, and therefore the reduction in drift. However, the decrease in maximum shear stress was unexpected as it was thought that the thicker reinforcement layers would have increased wallette strength.

Wallette four achieved an ultimate shear stress of 0.34 MPa, with the ultimate drift being 2.7%. The increase in shear stress was expected due to the thicker layer of ECC reinforcement. Also, any defects in the shotcrete layer that existed on W4 would be less significant when compared with thinner layers. As masonry surfaces are relatively rough and not flat, shotcreting 10 or 20 mm when measured from the border of the walls does not guarantee the same thickness throughout the entire wall. Some parts of the wall where the masonry protrudes may experience a significant reduction in reinforcement thickness and become weaker points in the reinforcement layer. Thicker overall layers of shotcrete, such as used in W4, reduce the chance of certain points having significantly less reinforcement than other areas, therefore providing a higher increase in shear stress over thinner layers. The maximum drift of W4 is comparable to the drift measured in W3. From the similar level of drift, it was concluded that for ECC reinforcement layers that exceed 20 mm, the failure mechanism became the delamination of the ECC layer. The delamination occurred as ECC to masonry bonding strength was unable to accommodate the shear strain that arose from the movement of the masonry relative to the ECC layer, which had an increased stiffness due to thicker layer of ECC when compared with W2 and W3. To improve the drift, either chemical or mechanical methods could be used to increase the bond strength between ECC and masonry.

W5 had an ultimate shear stress of 0.21 MPa and horizontal drift of 3.4%. The value of the ultimate drift was higher than the values measured for W3 and W4, and the increase in shear stress and drift was thought to be due to two reasons. Firstly, the control pins served as an anchorage, providing better bonding and shear stress transfer between the URM wall and the ECC reinforcement layer; secondly, the control pins provided a more consistent accuracy over the shotcrete depth as they display the necessary shotcrete depth every 500 mm in comparison with other wallettes where the depth was only measured at the edges. Therefore it is possible that the use of control pins as a method to specify the shotcrete depth is beneficial in terms of the performance of the reinforcement.

5 MASONRY OUT-OF-PLANE TESTING

In the event of an earthquake, eight out-of-plane failure modes for URM walls have been identified by D'Ayala and Speranza (2003). The study reported here focused on the failure mode that involves the development of horizontal cracks at wall mid height, followed by rocking and ultimately spalling of the wall when the ultimate displacements is reached (Griffith et al., 2004). The targeted failure mode generally occurs in URM walls that are simply supported and non load-bearing.

Two 4.1 m high by 1.2 m wide by 220 mm thick masonry walls (referred as WO1 and WO2) were constructed in order to test the out-of-plane response of URM walls reinforced with ECC shotcrete. It was anticipated that the ECC reinforcement would enhance the out-of-plane applied wall pressure required to displace the URM walls. The walls were tested as-built at 28 days. A steel frame was used as the reaction frame for the application of loads onto the walls, and air bags were used between the plywood attached to the frame and the masonry wall to provide a distributed load rather than a concentrated load. Four load cells connected the plywood frame to the steel frame to measure the force that was exerted by the air bags. No other connections existed between the plywood and steel frame. The air bags were inflated using existing air pumps in the laboratory with pressure control systems. The walls were supported at the top and bottom with steel angles clamping the bricks to the ground. The top of the brick wall was also clamped with steel angles to a concrete wall. No axial loads were applied on either of the walls, which represented either a one-storey URM wall or the top storey of a 2-storey URM building. The setup schematic is shown in Figure 16.

5.1 Testing observations and results

Air bags were gradually inflated to provide an increasing pressure onto the surface of the masonry wall. Pressure was applied until a clearly visible crack occurred, similar to that shown in Figure 17 (Derakhshan & Ingham, 2008).

For WO1, the crack occurred between the masonry joints at 3.08 m (75% of the wall height), with the maximum force recorded being 4.2 kN. The out-of-plane wall displacement at the crack location was 115 mm. When the pressure inside the air bag was released, the wall returned to its original position, and the crack closed up and was no longer visible. The reason that WO1 did not crack at wall mid-height was suspected to be due to the mortar produced by the mason being inconsistent, as the wall was constructed over 3 days. Both the top and bottom support of the wall remained rigid during the testing and therefore did not influence

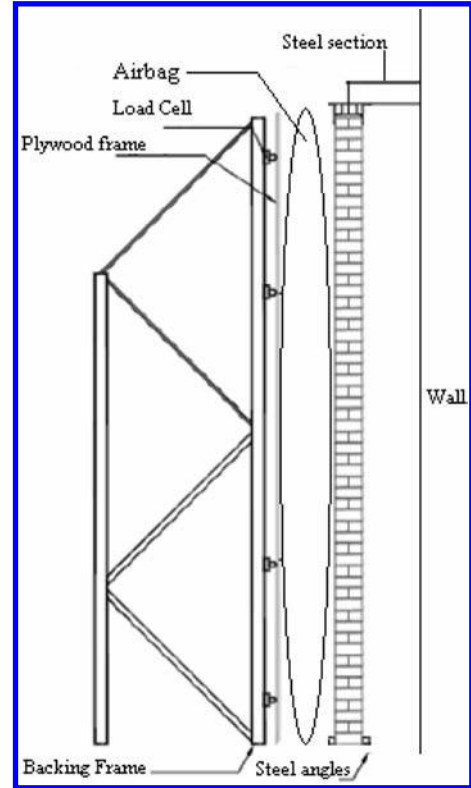


Figure 16. Setup schematic for out-of-plane wall testing.



Figure 17. Close up of the mid-height crack of the wall, reproduced from Derakhshan & Ingham (2008).

cracking formation. For WO2, a horizontal crack occurred at 1.9 m from the ground, at approximately mid-height. The maximum force recorded was 4.5 kN, similar to the value obtained in test WO1. The crack displacement was 18 mm. WO2 was not displaced to 100 mm because, prior to testing, the wall was already leaning towards one direction as the result of the workmanship of the mason. Therefore, it was determined that displacing the wall 100 mm could cause the wall to collapse. The results of the testing are shown in Figure 18.

Immediately after as-built testing, shotcrete was applied on one surface of each wall. For WO1, shotcrete was applied on the tension surface (referred as ECC-RT), and for WO2, shotcrete was applied on the compression surface (referred as ECC-RC). The thickness of ECC applied to both walls was 10 mm per layer in 3 layers. The walls were then re-tested 28 days after the application of shotcrete. The force-displacement response of the two walls is shown in Figure 19 and Figure 20.

It can be seen from Figure 19 that with 30 mm of ECC applied on the tension surface, the out-of-plane stress capacity of the URM walls did not increase significantly when compared with the

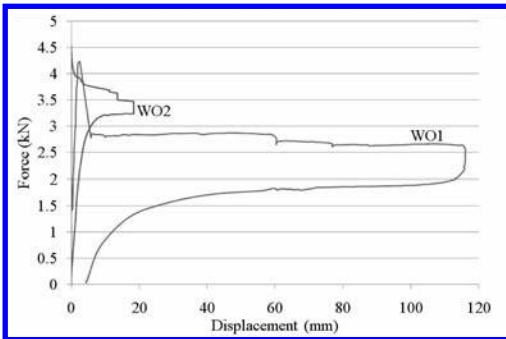


Figure 18. Force-displacement diagram of the as-built walls.

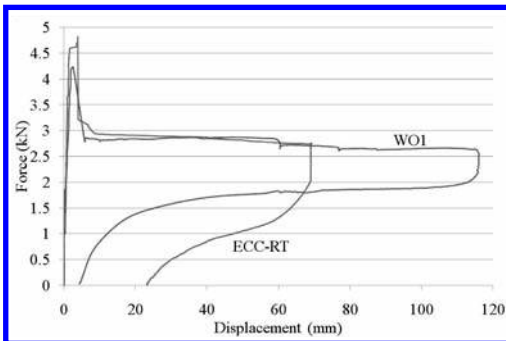


Figure 19. Force-displacement graph of ECC-RT.

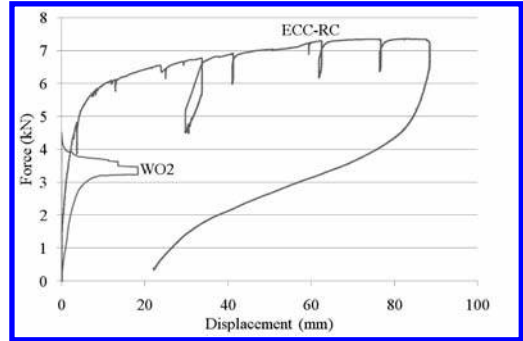


Figure 20. Force-displacement graph of ECC-RC.

non-retrofitted walls. The maximum load resisted by the wall was 4.8 kN. During testing a localised crack of the ECC reinforcement occurred at a height of 2.05 m. The cause of the localised crack was due to strain on the tension surface of the wall exceeding the tension strain capacity of the ECC layer. To increase the strain capacity of the ECC layer, additional reinforcements such as steel mesh will be required.

From Figure 20, it can be seen that ECC reinforcement was much more effective when sprayed on the compression surface of the masonry. The maximum load of ECC-RC was to 7.4 kN, which increased by a factor of 1.6 when compared with WO2. The wall was displaced to 87.7 mm at mid-height with no cracking observed on the ECC reinforcement layer. Two cracks were observed on the brick tension surface, but the wall was held together by the ECC layer. Because displacing the wall any further could cause wall collapse, the ECC reinforcement layer remained in the elastic range during the test.

6 CONCLUSIONS

A mix design to produce an ECC composite with 2% tensile strain was developed. It was determined that a successful ECC mix design requires emphasis to be placed on the sand used. Beach silica sand, with its rounder surface, is more favorable than dune sand. The average grain size of the sand should be as small as feasible to allow easier fibre slippage within the ECC matrix.

Three stack high single leaf masonry prisms were constructed in order to test the reinforcing effect of ECC on the shear capacity of unreinforced masonry. Energy absorption was significantly higher when compared with non-retrofitted prisms, indicating the suitability of using ECC for seismic retrofit.

Five 1.2 m by 1.2 m wallettes were constructed in order to investigate the in-plane diagonal shear capacity of masonry walls retrofitted with ECC. The brittle diagonal failure behaviour of conventional masonry wallettes was replaced by a ductile failure associated with the development of multiple diagonal cracks. The energy absorption increased significantly as for the previous prism test results.

Finally two 4.1 m out-of-plane URM walls were constructed in order to investigate their response to face loads. The walls were retrofitted with 3 layers of 10 mm of ECC, with one wall sprayed on the tension surface and the other having ECC sprayed on the compression surface. When applied on the tension surface the ECC reinforcement was not effective as the strain exceeded the tensile strain capacity of the ECC. When applied on the compression surface, the ECC reinforcement increased the wall's maximum load by a factor of 1.6 and still remained in the elastic range. It is concluded from the reported testing that ECC is effective as in-plane reinforcement for masonry buildings but that additional reinforcements is required for out-of-plane loading.

ACKNOWLEDGEMENTS

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REFERENCES

- ASTM, 2002, Standard Test Method for Diagonal Tension (Shear) in Masonry Assemblages. *American Society for Testing and Materials*.
- Bruedern, A.-E., Abecasis, D. & Mechtcherine, V. 2008. Strengthening of Masonry Using Sprayed Strain Hardening Cement-Based Composites (SHCC) Seventh International RILEM Symposium on Fibre Reinforced Concrete: Design and Applications, Chennai (Madras), India, 17–19 September 451–460.
- D'Ayala, D. & Spornaza, E. 2003. Definition of Collapse Mechanisms and Seismic Vulnerability of Historic Masonry Buildings. *Earthquake Spectra*, 193, 479–509.
- DBH 2004, Building Act 2004. In Department of Building and Housing—Te Tari Kaupapa Whare, M.O.E.D. & Government, N.Z. (Eds.). Wellington, New Zealand.
- Derakhshan, H. & Ingham, J.M. 2008, Out-of-Plane testing of an unreinforced masonry wall subjected to one-way bending Proc. 2009 AEES Conference Ballarat, Australia 21–23 November.
- Fillitsa, V.K. & Michael, N.F. 1992, Effectiveness of Seismic Strengthening Techniques for Masonry Buildings. *Journal of Structural Engineering*, 1187, 1884–1902.
- Griffith, M.C., Lam, N.T.K., Wilson, J.L. & Doherty, K. 2004. Experimental Investigation of Unreinforced Brick Masonry Walls in Flexure. *Journal of Structural Engineering*, 1303, 423–432.
- Kahn, L.F. 1984, Shotcrete Strengthening of Brick Masonry Walls. *Concrete International: Design and Construction*, 67, 34–40.
- Kanda, T., Saito, T., Sakata, N. & Hiraishi, M. 2003, Tensile and Anti-Spalling Properties of Direct Sprayed ECC. *Journal of Advanced Concrete*, vol. 13, no. 3, 269–282.
- Kim, Y.Y., Kong, H.-J. & Li, V.C. 2003, Design of Engineered Cementitious Composite Suitable for Wet-Mixture Shotcreting. *ACI Materials Journal*, 1006, 511–518.
- Li, V.C. & Fischer G. 2002, Reinforced ECC- An Evolution from Materials to Structures. *First fib Congress*, Osaka, Japan. 14–18 October.
- Li, V.C., Wang, S. & Wu, C. 2001, Tensile Strain-Hardening Behaviour of Polyvinyl Alcohol Engineered Cementitious Composite (PVA-ECC). *ACI Materials Journal*, vol. 98, no. 6, 483–492.
- Magenes, G. & Calvi, G. M. 1997, In-plane seismic response of brick masonry walls. *Earthquake Engineering & Structural Dynamics*, 2611, 1091–1112.
- Russell, A.P. & J.M. Ingham, 2008, Architectural characterisation and prevalence of New Zealand's unreinforced masonry building stock. *New Zealand Society for Earthquake Engineering Conference*, Wairakei, Taupo, New Zealand, April 11–13, 2008.

Sprayed concrete nozzle operator training and certification

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ABSTRACT: Quality sprayed concrete (shotcrete) is the product of a complex interaction of numerous factors. One of the key factors in achieving a cost effective, high quality concrete is the knowledge, skill and experience of the nozzleman. This paper discusses the state of play in the international sprayed concrete community with regard to nozzle operator training and certification, including a number of independent training and certification programs being developed or provided by organizations in Europe and America. The paper also presents a Queensland Government accredited training package for shotcrete operators which is competency-based, and includes theory and practical training. The training program is aimed at developing a core understanding of concrete operations in an underground environment, progressing from pump operator to nozzle operator and finally to shotcrete project management.

1 INTRODUCTION

1.1 Background

Shotcrete has been in use for just over 100 years with the first use being dry mix followed by wet mix after the second world war. It has a long history of use in a wide range of applications where the skill and competency of the nozzleman is recognized as one of the key factors in its success (other factors being preparation of substrate, use of suitable materials, mix designs, and equipment). This was recognized even in the early years and a lot of emphasis was placed on the training of the shotcrete nozzleman as a *craftsman* (ACI, 2008).

There are no universal standards for training and certification of nozzleman as yet, however there are a number of programs either in development or implemented across the globe.

1.2 Why train and certify nozzle operators

Sprayed concrete use is intended to achieve a certain placed quality which is integral to the performance of the project, be it a civil load bearing structure or surface support in mining. There are numerous standards and specifications for the design of concrete structures and for preparation of the concrete itself, however it is not enough to have good standards,

materials, mix design and equipment if the final placement is not carried out by a skilled operator.

A designer would never approve the building of a steel bridge with non-certified welders, yet until recently there was no such recognized independent certification for nozzlemen and even now the certification requirements are not consistent between projects.

Training and certification provided by independent institutions is a necessary requirement to ensure that concrete is placed reliably and to specification, such that quality is assured. Consistent, high quality application will give designers and project owners more confidence in using this method for cost effective development.

1.3 Shotcrete use and certification

Sprayed concrete is widely used in construction and mining. For example it can be found in:

- underground excavations, either for temporary or permanent support,
- soil nailing and slope stabilization,
- houses, swimming pools and various sports facilities,
- repair and reinforcement of concrete structures, for instance bridges, buildings,
- architectural and faux rock features, etc.

It is used for applications needing immediate safety and/or long-term durability. In all cases, one needs to be able to rely on the quality of the *in-situ* placement.

The method of applying sprayed concrete also depends on a number of factors such as cultural background (prevalent usage in the locality), expertise in application technique, equipment availability, logistics of supply, specific requirements of the project and costs. Generally sprayed concrete (shotcrete) is divided into two categories, being wet mix and dry mix. The shotcrete in both methods can either be applied manually or robotically.

The types of training and certification available do have common basic principles but must be distinguished as they require different expertise:

- The dry-mix process is different from the wet-mix process (the nozzle operator has to precisely regulate the water addition at the nozzle),
- Projection onto vertical surfaces is easier than projection onto overhead surfaces,
- Repairs by spraying grout in small galleries requires specific skills, and
- Robot-aided spraying requires different skills from manual spraying.

Some certification schemes distinguish between all these fields, others only some of them. Some offer several qualification levels to make a distinction between more or less skilled operators, while others do not. What is important is not to get a certificate but to get *the* certificate that corresponds to the requirement.

2 SHOTCRETE CERTIFICATION

2.1 Available certification programs

The information used in this chapter has been compiled through members of the International Tunneling Association Working Group 12 “Shotcrete Use”; and may not be complete. There are a few different types of certification organizations ranging from civil engineering companies and shotcrete contractors conducting in-house training and certification through to concrete associations and statutory bodies recommending or stipulating certification requirements.

2.1.1 Certification in North America

The American Shotcrete Association (ASA) was formed in 1998 by shotcrete users, practitioners, academics and suppliers to promote “the safe and beneficial use of the shotcrete process”. One of the first priorities of the ASA was to establish a nozzleman training and certification program. The ASA program was started with the understanding that if the American Concrete Institute (ACI) developed

a program then the ASA would withdraw its own and support the ACI.

In 1997 the American Concrete Institute established ACI Committee C660, Shotcrete Nozzleman Certification, with the mandate to develop, maintain and update programs for use in the certification of persons performing as shotcrete nozzle operators. The ACI Committee C660 developed a thorough certification program with strict policies, guidelines and procedures that responded to the needs of the shotcrete industry. It also vetted knowledgeable persons in the shotcrete industry to act as ACI approved examiners. This program was rolled out in 2001 and replaced the ASA program (Morgan & Dufour, 2009). This certification program is the most widely used with over 800 ACI certified shotcrete nozzle operators. It is intended to certify experienced nozzle operators (prerequisite: 500 hours of verified work experience as a nozzle operator or operator-in-training). It certifies both theoretical knowledge and practical ability. It distinguishes four categories of work:

- Wet mix process, vertical position only,
- Wet mix process, vertical & overhead positions,
- Dry mix process, vertical position only, and
- Dry mix process, vertical & overhead positions.

The examination sessions are conducted by local sponsoring groups of which there are two: the ASA, which administers examinations in English and Mexican Spanish, and Laval University, Quebec, Canada which administers examinations in French and English. The ASA requires the examination candidates to first take the ASA Shotcrete Nozzleman Training before sitting for the certification. This is to impart a consistent message regarding best shotcrete practice to all participants, and also to improve the chances of nozzle operators passing (there was a high failure rate in the written examination prior to this training initiative).

In addition to shotcrete nozzleman certification, many projects with structural shotcrete now also require pre-qualification of the entire shotcrete crew for the specified project by shooting and evaluation of a pre-construction mock-up. After shooting, the mock-ups are evaluated by coring or diamond saw cutting to expose intersecting reinforcing steel, so that the quality of the workmanship can be assessed. Cores can also be taken from locations without reinforcing steel, or separate plain shotcrete test panels, to prequalify the shotcrete mixture design supplied. Parameters evaluated can include compressive strength, boiled absorption and volume of permeable voids (ASTM C642, 2006).

2.1.2 Certification in France

A certification scheme has been available since 2001 in France from ASQUAPRO, a French

non-profit-making technical association for the “Quality of Sprayed Concrete and Mortars” (ASQUAPRO, 2009). This scheme is based on a Canadian certification program developed for the Ministry of Transport from Quebec (MTQ) in 1995–96. The Canadians have since worked with the ACI and now use that as the standard program.

As with ACI certification, the French also require both theoretical and practical knowledge to be tested. The major differences are as follows. The ACI prerequisite of 500 hours work is not necessary. Instead there are four levels of qualification; the first is aimed at operators with less than 100 hours experience who cannot be certified as nozzle operators but, if they pass both the theoretical and practical examinations, they can get “approval to spray” under the supervision of a certified nozzle operator. The three other certification levels correspond to increasing degrees of operating experience: “certified”, “confirmed”, “highly qualified” operators.

Dry and wet mix processes are distinguished but not vertical and overhead projection: the nozzle operator must spray both vertical & overhead panels. Specific certifications have been established for spraying grout in small galleries, and more recently for robotic application. The certification can be taken either at the end of a training session conducted by independent institutes, or on the premises of the companies wanting to certify their nozzle operators or on the building site.

2.1.3 *Certification in Norway*

The Norwegians have a “Concrete Educational Council” which is an industry-based association affiliated to the Norwegian Concrete Association. The Council offers both training (course content and competent trainers) and certification (examination, certificates and registration).

The competency requirements for key personnel are specified in the Norwegian Standards. The team leaders (foreman; production, control and testing) have to show that they are familiar with sprayed concrete. For underground operations the predominant application is via wet mix robotic application for which the operator is considered a crew leader and as such, must be certified.

Dry mix operators engaged in concrete repair are considered members of the concrete repair crew and do not need certification, however their team leaders do. There are three types of certification covering both manual and robotic spraying, for:

- Foremen and crew leaders
- Normal class production leaders and control & testing leaders
- Extended class production leaders and control & testing leaders.

There is no distinction between vertical and overhead position. There is no practical examination but practical experience is the first prerequisite for certification, comprising: one year for foremen, crew leaders and normal class production or control & testing leaders; or three years for extended class production and control & testing leaders. The second prerequisite is education, comprising: engineering education for production leaders and leaders of control & testing, professional education for foremen and crew leaders. The last requirement to pass the Norwegian certification is a 5-day course (or 2×2.5 days) at the end of which all participants take the same exam.

2.1.4 *Certification in Germany*

In Germany, a certificate is specifically required for the repair/strengthening of reinforced concrete structures by specific sprayed concretes or mortars containing resin. This certificate is issued after a training session by academies authorized by the “German Concrete and Civil Engineering Association”. It consists of two parts, theoretical and practical (spraying concrete on a vertical surface with a high content of reinforcement). Spraying is carried out by the manual process only. It can be done either by wet or dry process. There is one single certification level, dedicated to experienced nozzle operators.

The German Standard on shotcrete, DIN 18551 (2005), requires the nozzleman to have sufficient experience and knowledge for all applications but does not mention a specific certificate.

2.2 *Certification programs under development*

2.2.1 *Brazil*

Brazil has had a certification program since 1996 (Vieira et al, 1999) and some certifications have been issued but no specific institution is responsible and the number of certifications is unknown. The Brazilian Concrete Association (IBRACON) is working to achieve nozzle operator certification as part of the Brazilian federal program of workmanship qualifications.

The certification will first focus on the dry mix process but the program could be extended to wet-mix in the future. There will be a distinction between manual and robotic projection but vertical and overhead spraying will probably not be differentiated. Examinations will cover both the theoretical and practical aspects. There will be only one certification level.

2.2.2 *EFNARC*

The European Federation of Specialist Construction Chemicals and Concrete Systems (EFNARC) was founded in 1989 and has a wide ranging membership including manufacturers, consultants,

academics and contractors. An EFNARC working group is preparing a nozzle operator certification scheme. They plan to deliver a document of about 200 slides for self-study, in 2009. The certification will cover the field of robotic projection of wet concrete for rock support, including vertical and overhead projection, as well as spraying into lattice girders and tunnel profiles.

The EFNARC certification scheme begins with examiners' certification during sessions organized by VSH in the test gallery at Hagerbach. The duration of the examiners' course will be 2 or 3 days. The second step will be the local certification of nozzle operators (on site) validated by approved examiners after providing a nozzle operator course in the local language. The theoretical exam will be a 50-question multiple choice paper. The practical exam will consist of spraying concrete with a robot and testing its consistency. There will also be oral and visual examinations.

2.2.3 Sweden

The Swedish Shotcrete Center and Vattenfall Utveckling, consultants and Research & Development organisations, commenced certification of nozzle operators in Sweden in 2006. The certification examination was theoretical and written. Since then, a new working group has developed a programme for certification of supervisors and nozzle operators. The working group is drawn from mine workers, contractors, the Swedish National Road Department, the Swedish National Rail Department, suppliers, Vattenfall Research and Development, and others.

Examinations will be theoretical and practical, and they will be taken after a 40 hour training course, called "rock support and repairing with shotcrete". The education before certification will be the same for supervisors and nozzle operators and therefore there will be only one certification level but different requirements for prior knowledge. Certification will not distinguish between vertical and overhead position, manual and robotic spraying.

Table 1. Certification relative to application method (after Larive, 2009).

Entity/ Location	Process		Type of spraying	
	Dry mix	Wet mix	Manual	Robotic
ACI	✓	✓	✓	
ASQUAPRO	✓	✓	✓	✓
Norway	✓	✓	✓	
Germany	✓	✓	✓	
Sweden	✓	✓	✓	✓
EFNARC		✓		✓
IBRACON	✓		✓	

Table 2. Certification relative to certification level and training (after Larive, 2009).

Entity/ Location	Level of certification		Training	
	One level	Several levels	Pre-requisite	Separate
ACI	✓			✓
ASQUAPRO		✓		✓
Norway		✓	✓	
Germany	✓		✓	
Sweden	✓		✓	
EFNARC	✓		✓	
IBRACON	✓			✓

2.3 Summary of certification programs

The types of certification and relationship to application and training are summarized by Larive (2009) and are reproduced below (Tables 1 & 2). Larive & Gremillion (2007) provide more information on the contents of examinations; pass requirements, panel assessments, practical tests, duration of validity and re-examination conditions.

3 TRAINING

As discussed above, training is provided in North America by ASA as a prerequisite for the ACI certification. In Europe independent institutions such as CPO (training organization near Eperon, France) and ABCCR (training organization in Port-Marly, France) provide training in parallel with the ASQUAPRO produced publications. The Norwegian, German and Swedish certification schemes require attendance at training sessions or study of training material. The Brazilians plan to separate the two aspects of training and certification with IBRACON in charge of certification and other institutions providing training. The vast bulk of the theoretical and practical training being conducted around the world is through equipment suppliers and admixture manufacturers (Normet, Jacon, BASF, TAM, SIKA etc). This is particularly relevant for the wet mix robotic process for tunneling and mining projects where such courses are tailored for the client's requirements.

A number of shotcrete contractors in mining (Jetcrete and Macmahon in Australia) also have structured training programs for their crews. These are usually carried out on the worksite. Civil engineering companies seeking to raise standards also conduct training and certification (Hendrix, 1983). Some of this certification is in house (Morgan Est, UK) but others are through an independent body (Freyssinet in France).

3.1 *A shotcrete training and certification program in Queensland, Australia—case study*

3.1.1 *The training program*

A Queensland Government accredited training program has been developed by Stratacrete Limited for robotic application of wet mix shotcrete. This program has been developed to assist shotcrete operators working in the mining industry to meet or exceed nationally recognized standards of competence established for this skill. It has been designed in accordance with competency-based training concepts and relates directly to Certificates (II to IV) and Diploma in Metalliferous Mining Operation (DEST, 2005).

All learning material, assessment instruments, activities, and related resource documents covered in this program relate to the application of shotcrete by mine personnel. Changes to equipment, company operating or mining procedures, policy, or production techniques are reflected in the instructional material. This program has been designed as an instructional resource comprising five levels (Loncaric & Donnelly, 2009):

- Level 1—Trainee Shotcrete Operator,
- Level 2—Pump Operator,
- Level 3—Nozzle Operator,
- Level 4—Advanced Nozzle Operator (or Trainer-Assessor),
- Level 5—Project Manager.

The requirements for the training are implemented in accordance with Australian Quality Training Framework standards for RTOs (Registered Training Organizations). Any substantial alteration to the assessments, resource material, or management of the program without the approval of the RTO will void the quality assurance.

The training program involves both theoretical (classroom based) and practical (on the job) training with written reference information provided. The following list of topics is covered in the training. The topics have been grouped to provide a staged progression from novice to expert operator. The first two levels (Tables 3 & 4), when successfully completed, will meet the knowledge requirements of the MNM20205 Certificate II in Metalliferous Mining Operations (Underground). The third level (Table 5), when successfully completed, will meet the knowledge requirements of the MNM30205 Certificate III in Metalliferous Mining Operations (Underground).

The fourth level (Table 6), when successfully completed, will meet the knowledge requirements of the MNM40205 Certificate IV in Metalliferous Mining Operations (Underground).

The fifth level (Table 7), when successfully completed, will meet the knowledge requirements of MNM50105 Diploma of Metalliferous Mining (Open Cut and Underground).

3.1.2 *Assessment*

The learning outcomes are confirmed through the assessment instruments provided with this training program. Trainees will be deemed to have acquired the desired competency standard when they can demonstrate to an accredited assessor that they can safely and efficiently apply shotcrete in accordance with the Performance Criteria identified in the appropriate Competency Standards.

RCC (Recognition of Current Competencies) and RPL (Recognition of Prior Learning), are ways of recognizing that a person has acquired the necessary competencies through previous learning or

Table 3. Level 1—Trainee shotcrete operator.

Description	Estimated time
Overview of training program	2 hours
QMITAB generic metalliferous core induction program	10 hours
QMITAB generic metalliferous elective induction program	5 hours
Introduction to shotcrete and illustrated glossary	2 hours
Underground communication	2 hours
Plan and organise individual work	2 hours
Hazard identification and risk assessment	3 hours
Site induction	30 hours
Site light vehicle ticket (equipment manual)	10 hours
Concrete basics, including fibre	3 hours
Batch plant operator (theory and quality control)	3 hours
Site agitator ticket (equipment manual, formal training component)	30 hours
Shotcrete Rig (equipment manual, formal training component)	40 hours
Maintenance and minor repairs	10 hours
Consolidation of skills and allowance for scheduling	30 hours

Table 4. Level 2—Pump operator.

Description	Estimated time
Job safety analysis	3 hours
Crew communication	3 hours
Introduction to ground preparation	10 hours
Maintenance trouble shooting	10 hours
Electrical and hydraulic trouble shooting	10 hours
Mix trouble shooting	10 hours
Factors affecting mix performance	5 hours
Fibre balls	2 hours
Extra water	2 hours
Oversize aggregate	2 hours
Admixtures and dosages	5 hours
Quality implications	5 hours
Rejection of loads	5 hours

Table 5. Level 3—Nozzle operator.

Description	Estimated time
Supplier communication	5 hours
Site specific ground awareness course	10 hours
Australian Standards in testing	10 hours
Certification to spray test panels	30 hours
Log of fifty test panels to required standard	40 hours
Understanding of tensile strength	5 hours
Flexural strength	5 hours
Shear strength	5 hours
Bond strength	5 hours
Develop standard operating procedures	5 hours
Senior first aid certificate	15 hours
Advanced admixtures and dosages	5 hours
Practical application of stabilizers	5 hours
Practical application of accelerator	5 hours
Practical application of water reducer	5 hours
Calculations of dosage rates by weight of cementitious material	10 hours
Justifying rejection of loads	5 hours
Limits of admixture quantities	5 hours
Consolidation of skills	40 hours

Table 6. Level 4—Advanced nozzle operator.

Description	Estimated time
Supervisory communication	10 hours
Leadership and motivation	10 hours
Introduction to contract management	10 hours
Supervisor for up to 24 hour, 7 day operation	40 hours
Parts of Certificate IV Workplace Training and Assessment	30 hours
Consistent procedures (quality control)	15 hours
Implement and maintain standards on site	10 hours
Consolidation of skills	10 hours

Table 7. Level 5—Project manager.

Description	Estimated time
Client communication	10 hours
Advanced contract administration	10 hours
Advanced site administration procedures	10 hours
Consistent procedures across all sites	10 hours
Supervisory level in charge of a 24/7 operation with 4 crews	10 hours
Hiring procedures	10 hours
Retrenchment procedures	10 hours

experience. Regardless of how the particular skill was acquired, there is provision in the training structure to recognize this skill and provide the appropriate qualification. By providing a Statement of Attainment for competencies completed, the employee gains a skill that is recognized in any metalliferous

underground operation in Australia. As each competency is completed, the employee is one step closer to a qualification recognized under the Australian Qualification Framework (AQF) (DEST, 2007).

3.1.3 Case study on improvements due to training

3.1.3.1 North Parkes Mine

North Parkes Mine is a block caving operation that was undergoing infrastructure development where shotcrete was part of the long term ground support. There were a few issues regarding excessive re-work and poor practice, mostly due to loss of experienced staff and training limitations. Fortunately the site had good monitoring systems in place which could be used to measure shotcrete quality before and after training. After three months of training on a single shift basis the results shown in Figures 1 to 3 were achieved.

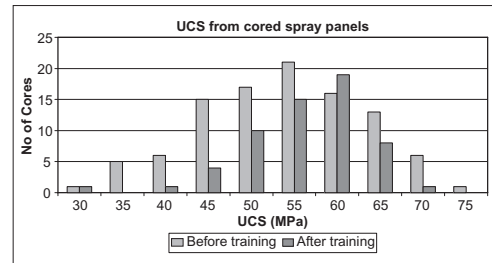


Figure 1. Improved UCS results after short term training.

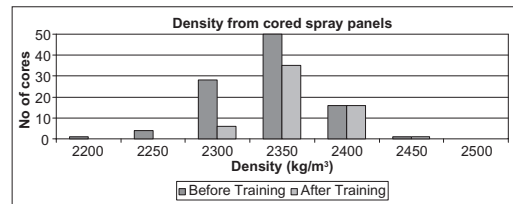


Figure 2. Improved density results after short term training.

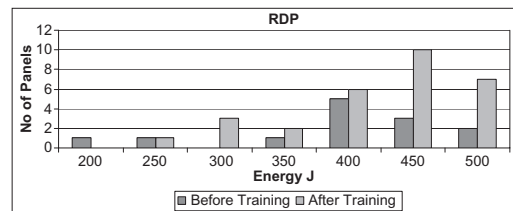


Figure 3. ASTM C1550 round panel energy absorption at 40 mm.

Note the mean strength increased from 55 MPa to 60 MPa and the distribution of results went from a Normal curve to a curve more skewed towards higher strength. Accelerator usage was specified at less than 21.5 L/m³. Before training, accelerator was applied within specifications 65% of the time. After training it was applied within specifications 85% of the time. The in-situ density improved after training with the mean increasing slightly from 2340 kg/m³ to 2360 kg/m³ and the distribution showing less scatter (Figure 2). The ASTM C1550 round panel tests also showed improvement with the mean energy absorption rising from 400 Joules to 450 Joules (Figure 3).

One of the significant improvements the trainers noticed was the development of an ability by the shotcrete operators to plan how they would spray a heading or a particular area that required rehabilitation. This resulted in significant improvements in uniform thickness and reduced fall out. Unfortunately rebound including fallout was not measured or calculated for this short term training program. The use of depth stamps also improved quality assurance on thickness (Figure 4).

3.1.3.2 Peak Gold Mine

In this example the mine owner-operators were re-starting their shotcrete system after resolving supply issues. The mine decided to set up a temporary batch plant and to return employees from other duties to shotcrete operations. The mine had also recently purchased and commissioned a new Maxijet shotcrete rig.

After implementing a short term training program on site the following results were achieved:

- Increase in average UCS from 32.5 to 35 MPa (7.6% rise),
- A saving in excess of 40 litres per heading in accelerator usage,
- There was a measurable improvement in the quality of the shotcrete applied underground,

- There was a measurable improvement in the performance of the shotcrete operators,
- Improved operator understanding of the use and control of the Maxijet including concrete pump speed and pressure, accelerator pump setting and engine speed setting.

3.2 Recommendations for implementing a training program

Before a training program can be devised, an initial audit of the shotcrete system on site should be conducted involving all parties (including suppliers) who contribute to the shotcrete system. Ideally this would require 6 to 8 weeks to allow for the collection of suitable data. This data will ideally consist of:

- UCS test cylinders at the batch plant,
- Test panels sprayed underground for density and operator UCS tests,
- Panels sprayed underground to determine energy absorption via the ASTM C 1550 round panel test or the European square panel test EN 14488-5,
- Thickness stamping or drilling to determine applied thickness and consistency,
- Accelerator usage measured at the pump and reconciled to what the mine can measure,
- Rebound measured for each operator or calculated by comparing shotcrete volumes batched to shotcrete volumes placed.

Collecting data can result in improvements in the shotcrete system simply because personnel are now being introduced to existing or new procedures for placing shotcrete, that were not previously well known or established on site.

The site can then choose to either implement its own training program or to bring in external consultants and expertise. Before training can be implemented a knowledge-base must be established and skills-base accessed. The quality of the knowledge and skills captured will determine the success of the training program. To implement the program, time and resources will have to be allocated to ensure personnel are afforded a fair opportunity to develop their shotcrete skills and knowledge. The shotcrete and training system must be monitored to ensure there is: no loss of core skills or knowledge as personnel are promoted or move away; no obsolescence in procedures as equipment, materials, chemicals and legislation are developed; and inadequate ground support or unmet legal requirements. Once a stable training program and shotcrete system are in place it becomes much easier to establish the benefits of optimizing the mix design, faster setting accelerators, curing sprayed

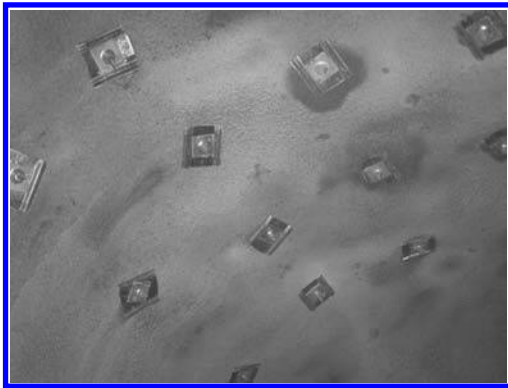


Figure 4. Thickness Stamps (note “X” shaped stamps).

shotcrete, changing equipment, use of different fibers, and controlling thickness.

4 CONCLUSIONS

There is a significant gap between the critical impact that the nozzle operator has on the quality of sprayed concrete and the independent training and certification that is available. This is recognized as a concern by sprayed concrete associations and there is currently a strong demand and effort to implement training and accreditation. The leaders in this area are presently the ACI and ASA in North America who have widely recognized certification programs for nozzle operators. The Europeans are taking a similar approach though ASQUAPRO who distinguish their certification on four levels ranging from “operator-in-training” to “highly-qualified” nozzle operator. Norway and Sweden place emphasis on certifying all staff involved in shotcrete, not just the nozzleman. Germany emphasizes skills for repair/reinforcement of reinforced concrete structures.

The Brazilian and the EFNARC certification schemes will, at least in the beginning, be dedicated to specific fields of application: the dry-mix process for the first scheme and the wet-mix process with robotic spraying for the last.

In Australia, a few shotcrete contractors conduct structured training programs in-house. One contractor, Stratacrete, has achieved state government accreditation and is providing independent training to mining companies and contractors. This training has had a significant impact on shotcrete performance and costs.

If the sprayed concrete industry can demonstrate, consistent and quality placement of shotcrete by certified nozzleman, designers and project owners will have increased confidence in using this method of placement. This will benefit the spraying industry and will also lead to efficiencies in the use of sprayed concrete. Therefore it is imperative that shotcrete certification be based on quality training and transparent assessment which is monitored by independent organizations.

REFERENCES

- ACI Committee E703. 2008. Shotcrete for the craftsman, CCS-4 (08), viewed on 31/07/2009, <http://www.concrete.org/bookstorenet/ProductDetail.aspx?SACode=CCS408>
- ASQUAPRO. 2009. Sprayed concrete use; Sprayed concrete implementation; Sprayed concrete mix design optimisation; Sprayed concrete control; Sprayed concrete design, viewed on 31/07/2009, <http://www.asquapro.asso.fr/Pages/publication.htm>
- ASTM Standard C642-06. 2006. “Standard Test Method for Density, Absorption, and Voids in Hardened Concrete,” ASTM International, West Conshohocken, PA, 2003, DOI: 10.1520/C0033-03, www.astm.org
- CEN/TC 104/WG 10, NA005-07-10 AA, 2005. DIN 18551 Shotcrete—Specification, production, design and conformity. Viewed on 31/07/2009, <http://www.nabau.din.de/cmd?workflowname=dinSearch&languageid=en>
- DEST (Department of Education, Science and Training). 2005. MNM05 Metalliferous Mining Training Package. Australian Training Products Ltd, Melbourne, Victoria, Feb. 2005. Online access: <http://www.atpl.net.au>
- DEST (Department of Education, Science and Training). 2007. AQTF 2007 Standards for Accredited Courses, Department of Education, Training and Arts, Commonwealth of Australia.
- Hendrix, R. 1983. Training for Shotcrete Nozzlemen and Tank Builders, publication #C830851, Copyright © 1983, The Aberdeen Group.
- Larive, C. 2009. Why and how to certify sprayed concrete nozzle operators? *Shotcrete for Africa conference*, 2–3 March, 2009. ITA, SAIMM & SANCOT, Johannesburg, South Africa.
- Larive C. & Grémillon K. 2007. Certification of shotcrete nozzlemen around the world, 33rd ITA-AITES World Tunnel Congress, May 5–10, 2007, pp. 1396–1399. Prague, Czech Republic.
- Loncaric, A.J. & Donnelly, B. 2009. Proposed training program for shotcrete operators. In Dight, P. (ed.), *SRDM2009, 1st Intern. Seminar on Safe and Rapid Development in Mining 2009*, Australian Centre for Geomechanics, Australia.
- Morgan, D.R. & Dufour, J.-F. 2009. Shotcrete nozzleman certification in North America. *Shotcrete for Africa conference*, 2–3 March, 2009. ITA, SAIMM & SANCOT, Johannesburg, South Africa.
- Vieira L.P., Jr. & Domingues de Figueiredo, A. 1999. Shotcrete—Standardization in Brazil. Eighth International Conference on Shotcrete for Underground Support, Sao Paulo, Brazil, April 11–15, 1999. pp. 36–45.

The use of Fibre Reinforced Shotcrete at Cadia Hill Open Pit

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ABSTRACT: The Application of Fibre Reinforced Shotcrete (FRS) at Newcrest Mining Limited's Cadia Hill Open Pit has been effective in reducing the exposure of personnel and equipment to rockfall events. This has been achieved through applying Fibre Reinforced Shotcrete to the infill of large structures as well as to areas of poor rock mass in order to contain material that may unravel over time.

1 INTRODUCTION

Newcrest Mining Limited's Cadia Hill open pit is a large scale, low grade gold and copper mine located approximately 20 km South West of the city of Orange in the Central West of New South Wales, Australia. The mine was commissioned in October 1998 and produces approximately 300,000 oz of gold and 25,000 tonnes of copper annually. Around 50 million tonnes of material is extracted from the pit per year of which 18 million tonnes of ore is processed in the ore treatment plant.

The Cadia Hill pit utilizes conventional truck/shovel bench mining that incorporates a fleet of 26 Caterpillar 793C rear dump trucks, one Komatsu PC8000 face shovel and two Komatsu WA1200 front end loaders. Benches are blasted and excavated in 15 m fliches with final batters having a height of 30 m. In areas that exhibit poor geotechnical conditions single 15 m high batters are incorporated. Conventional bulk drilling and blasting techniques are employed. The pit is approximately 1.6 km in diameter and 600 m deep. The pit will reach a final depth of approximately 700 m vertical once the third and final cutback is mined to completion.

1.1 Geotechnical environment

The Cadia Hill pit has four distinct rock types. These are Volcaniclastics, Monzodiorite, Sediments, and Monzonite. The Monzonite forms the bulk of the lower half of the pit excavation and has a UCS of up to 150 MPa. The Volcaniclastics, Monzodiorite, and Sediments exhibit UCS values of between 80 MPa and 130 MPa.

The Cadia Hill pit is a heavily structured pit with approximately 40 major (inter-bench scale) structures. Some of these structures can be classed as regional and extend for several kilometers. The rock mass in between these major structures varies greatly with Geological Strength Index (GSI)

values varying from approximately 50 to 90 and as low as 30. The areas of poorer rock mass are generally characterised by intense local jointing. In order to control the risk of rockfalls occurring within the operating areas of the pit, various forms of ground support are used. These include shear pins (to maintain berm catch capacity), mesh drapery, rock catch fences, and the application of Fibre Reinforced Shotcrete.

2 USE OF FIBRE REINFORCED SHOTCRETE

Fiber Reinforced Shotcrete (FRS) has been successfully used at the Cadia Hill pit as an effective tool for ground control and to ultimately reduce the risk of exposure of rockfalls to personnel and equipment working within the operating areas of the mine. Specifically, FRS has been used as a ground control measure in the following areas:

- The sealing of major regional scale structures that exhibit thick (greater than 1 m wide) clayey infill;
- Controlling areas of poor rock mass where unraveling may occur over time; and
- Controlling areas of poor ground located above and below main ramps which may compromise access should unraveling occur.

The FRS is typically applied by underground Jacon Maxijet spraying machines with 9 m reach booms. This is due to their ready availability from the nearby underground operations located within the Cadia Valley province. More recently, however, a much longer 30 m boom spray unit has been utilised. The FRS mix consists of 40 MPa concrete with a dosage of 8 kg of macro-synthetic fibres per cubic metre of concrete. The macro-synthetic fibres were the 48 mm Barchip Shogun brand with a tensile strength of 550 MPa.

2.1 *The sealing of major (regional scale) structural exposures*

Some major structural exposures within the Cadia Hill pit have infill thicknesses in the order of 1.0 metre to 1.5 metres across. This infill material is often heavily sheared rock flour and clay. Once this infill material is exposed within the pit excavation it will be in constant contact with weathering elements such as rainfall, runoff, wind, and variations in diurnal and seasonal temperature. All of these elements will have a degrading effect on this infill material which can in time cause unraveling and failures which can present a rockfall risk to personnel and equipment working within the operating areas of the mine. In order to control the unraveling of the infill material exhibited within these major structural exposures, the application of FRS to the surface of this infill material has been successfully used at Cadia Hill.

At the Cadia Hill Pit 15 m high batters are used in relatively weak rock masses while 30 m final batters are employed in stronger rock masses. This height of 30 m is beyond the reach of standard underground Maxijet spray units and FRS is therefore more effectively applied by long reach concrete booms or by hand-held spraying from an elevated work platform. In one specific case at Cadia Hill where a 1.5 m thick structural infill associated with the Baroness-Fish Tail structure was to be sealed with FRS in a 30 m high batter exposure, the use of hand-held spraying techniques was used. Hand-held spraying at heights of up to 30 m has its challenges. These include the following:

- Blasting inertia from the hose nozzle as FRS is sprayed. This will act to spin a free hanging man-basket around an axis making it impossible to effectively apply the FRS;
- Substantial weight of the main hose. A hose full of wet FRS up to 30 m long suspended from an elevated work platform is extremely heavy and cannot be supported by personnel; and
- Operator skill. Hand-held nozzle work is difficult with regards to applying a consistent layer of FRS and requires operators with substantial experience.

When engaging in the hand-held spraying technique at Cadia Hill for sealing the infill of the Baroness-Fish Tail structure at a height of up to 30 m from the ground, the following practices were used to counter the above listed challenges:

- A fixed elevated work platform was used rather than a free hanging man-basket. This eliminated the possibility of the platform spinning around as FRS was sprayed;
- A crane was used to lift and support the 30 m long length of hose suspended off the ground; and

- Skilled and experienced operators with prior experience in hand-held spraying were brought in for the task.

Altogether approximately 50 cubic metres of FRS was sprayed within the 1.5 m thick infill exposure of the Baroness-Fish Tail structure. In some areas of particularly poor infill material the thickness of the FRS was up to 500 mm. Figure 1 displays the process used in sealing this major structural intersection within the Cadia Hill Pit.

In order to avoid the potential for ground water to build up within the structure and pressurise the FRS layer, a small buttress of fragmented rock material approximately 3 m high was left on the berm flush with the batter face to allow water to drain. Figure 2 displays the Baroness-Fish Tail structure after sealing with FRS complete with self draining buttress located at the base of the structural intersection.



Figure 1. Hand-held spraying of the Baroness-Fish Tail structure.



Figure 2. Completed sealing of the Baroness-Fish Tail intersection with buttress at base.

2.2 Controlling areas of poor rock mass where unraveling may occur

In typical large open pit environments, many operating areas are subjected to the risk of rockfalls. These are primarily due to the unraveling of batter faces and crests that have relatively poor rock mass. The application of FRS to such batter faces can form a screen which helps contain the potential for these batter faces and crests to deteriorate and yield rockfalls.

At the Cadia Hill Pit, batters which exhibit significant jointing and weathered material (typically GSI values of 30–60) and are located in areas where personnel and equipment are operating in proximity below, are targeted with the use of FRS to provide a screen which will aid in containing any rocks that may unravel over time. Such areas are typically sprayed with 100 mm thick FRS in a continuous layer over the entire weak area. This practice has been effective in reducing the rockfall potential generated from these batters. Figure 3 displays areas on the lower South Wall at a depth of approximately 500 m that exhibit poor rock masses which have been successfully screened using FRS.

2.3 Controlling batters located above and below ramp to improve ramp stability

Ramps within large open pits provide critical access points to operating benches where the mining of ore takes place. If a ramp is compromised by rockfalls from above or by unraveling from below, critical access to these operating areas can be compromised and production and mine revenue may be impacted. The use of FRS at the Cadia Hill Pit has been successful in aiding the stability of the main ramp.

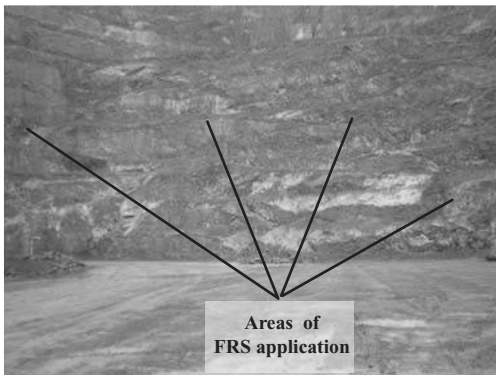


Figure 3. Fibre Reinforced Shotcrete application to areas of poor rock mass located above operating areas.

One such portion of the lower CB2 main ramp along the West Wall at approximately the 375 RL (approximately 550 m deep) was severely damaged during the firing of a routine trim shot. Blast energy was transmitted along continuous joint sets within the rock mass constituting the main ramp causing significant heaving and cracking along the ramp surface. Upon excavating the trim shot it was clear that the rockmass directly below the main ramp had been disturbed and that possible unraveling of this rockmass was likely to occur over time.

A layer of FRS approximately 150 mm thick was applied to the batter directly below the main ramp in order to control any unraveling that may occur and hence attempt to stabilize the running surface of the main ramp. Prisms were installed along the top edge of the ramp to determine if any unacceptable deformation was occurring even with the FRS screen in place as this section of ramp continued to be used for access to CB2. Over the months that followed this batter located directly below the main ramp remained stable with no unraveling occurring. This also allowed the successful mining of CB2 to occur unhindered by the need to preserve the stability of this section of ramp. Figure 4 displays the layer of FRS used to control unraveling directly below the main ramp at the 375RL.

2.4 Recent developments in application of FRS

In order to remove personnel and equipment from working in close proximity to batter faces while applying FRS, the use of underground Maxi-jets has been replaced with a much larger surface spraying unit. The make and model of this unit is a Schwing EPL 801R 29 m Swing Boom Concrete Pump. This unit has a boom that can extend to 29 m which makes spraying high batters considerably



Figure 4. Stabilisation of the 375RL ramp—CB2 excavation.



Figure 5. A 29 m boom purpose built surface spraying unit (Schwing EPL 801 R).

easier as well as allowing personnel and equipment to be located further away from the batter faces. Figure 5 illustrates this unit.

2.5 *Opportunities for improvement and research*

All applications of FRS were based on the success of previous experience. A significant opportunity exists for research into the formulation of an engineering design that determines the thickness of FRS required to prevent blocks unraveling from a batter and to hold together weak rock masses or in-fills. Techniques such as 3D Photogrammetry

can be used to successfully analyze a batter to determine the dimensions of the largest possible block. Laboratory tests can be conducted to determine the FRS thickness criteria and relevant engineering design methods can be developed to identify where the application of FRS is more efficient than other support systems.

3 CONCLUSIONS

FRS can be successfully used in large open pit environments to reduce the risk of rockfalls and control the unraveling of batters and crests that have a relatively poor rock mass. Specifically, FRS can be successfully used for the following: to seal the infill of major structural intersections to prevent unraveling of infill material; to form a screen over batters that exhibit poor rockmass in order to reduce the rockfall potential in areas that are located directly above operating areas; and, to stabilize batters located above and below main ramp intersections which can compromise ramp surfaces if unraveling occurs over time.

REFERENCE

Lowther, R. 2008. Cadia Hill Pit FRS Reconciliation.

Structural shotcrete in Western Canada

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ABSTRACT: Shotcrete is increasingly being used in Western Canada for construction of all or parts of reinforced concrete structures in lieu of conventional formed and poured cast-in-place concrete construction. The reasons for this trend are many and various, but include: use of single sided formwork, or in below-grade perimeter wall construction, elimination of formwork entirely. This commonly results in an accelerated reinforced concrete construction schedule and hence more cost effective projects. This paper, using case history examples, looks at the structural shotcrete design and construction process, including specifications, mixture designs, formwork and reinforcing steel installation and shotcrete supply, application, finishing and curing. Quality assurance and quality control inspection and testing requirements are also examined.

1 INTRODUCTION

The use of wet-mix shotcrete in lieu of conventional cast-in-place formed concrete, for construction of reinforced concrete walls has a long history of successful use on the West Coast of the USA. The use of wet-mix structural shotcrete has its origins in seismic retrofit of schools and commercial buildings in the Los Angeles and San Francisco areas in the 1950's, later spreading to Seattle and San Diego.

When carried out by qualified shotcrete contractors, the method proved to be technically sound and economically attractive compared to conventional cast-in-place concrete construction and subsequently found extensive use in the construction of permanent deep reinforced structural walls in in-the-ground parking structures and similar applications. This technology has, however, only recently moved across the 49th parallel into Canada and a number of four to five storey deep in-the-ground parking and transit and other structures are now being constructed in Vancouver, BC and Calgary, Alberta using wet-mix structural shotcrete. In addition general contractors on a variety of projects have found the shotcrete construction process to be so cost effective that they have elected to use it for construction of above-grade reinforced shotcrete walls, pilasters and other structural elements. Figure 1 shows an example of a high rise reinforced concrete building recently constructed in Vancouver, BC in which both the below and above-grade structural walls were constructed with wet-mix shotcrete. Of note is the very high quality of architectural finish that can be achieved with the shotcrete construction process.



Figure 1. High rise building in Vancouver with structural walls constructed by wet-mix shotcrete process.

2 SPECIFICATIONS

2.1 General

Fundamental to the construction of quality structural walls and other elements using the wet-mix shotcrete process is the provision of a suitable specification and provision of a suitable quality assurance (QA) and quality control (QC) inspection and testing program to ensure that the requirements of the specification are satisfied. If this is

not done, a less than satisfactory end product can result. Morgan and Totten (2009) provided a guide *Structural Shotcrete Specification* which could be readily adapted for any specific project.

2.2 Qualifications

Important to the construction of quality structural shotcrete works are the qualifications of the contractor and shotcrete crew. The referenced guide specification requires the use of a shotcrete crew foreman with at least five (5) years experience in construction of similar shotcrete projects. In addition, there is a requirement that all nozzlemen used on the project be certified by the American Concrete Institute to ACI CP-60 (09) for application of shotcrete to vertical surfaces using the wet-mix shotcrete process. The guide specification also requires the nozzlemen to have at least three (3) years experience of shooting on projects of similar size and character. These requirements are very important, as less than satisfactory construction has been experienced when less qualified contractors, or inadequately trained and experienced nozzlemen have tried to build reinforced structural shotcrete walls and other components.

2.3 Preconstruction mock-ups

In addition to the above contractor and nozzleman qualification requirements, the contractor is typically required to build a preconstruction mock-up. The mock-up is characteristically constructed to represent the most heavily reinforced section of the wall (or other structural elements) and is used to demonstrate that the contractor, nozzlemen and finishing crew are capable of constructing work in which:

- All the reinforcing steel is thoroughly encapsulated in dense shotcrete, without excessive voids, shadows or entrapped rebound or overspray,
- The proper shooting and finishing sequence is used and the shotcrete element is free of sags, tears or delaminations,
- The shotcrete is finished to the specified line grade, tolerance and type of finish required by the architect,
- Proper methods are used to construct construction joints and any architectural reveals or other features.

After the mock-up has gained sufficient strength (usually 2 to 3 days) it is cored and/or diamond saw-cut to check the quality of encapsulation of the steel in shotcrete. Once accepted, the mock-up is typically kept on site and forms the standard for acceptance/rejection of future shotcrete work.

In addition to shooting the mock-up, a standard ASTM C1140 450 × 450 × 110 mm panel is shot for qualification of the shotcrete mixture. Cores

(75 mm dia) are typically extracted for testing for compressive strength at 7 and 28 days. In addition, specifications also require the plastic shotcrete to be tested for slump and air content at the point of discharge into the shotcrete pump and the *as-shot* air content. Determination of air content is important for shotcrete that will be subjected to cycles of freezing and thawing during service.

3 MATERIALS

3.1 Cementing materials

Most structural shotcrete used in buildings in Western Canada is made with a combination of a CSA Type GU (General Use) Portland Cement and CSA Type F or CI fly ash: CSA A3000-03. In structures requiring higher early compressive strength (say 20 MPa in 24 hours) or high later age strengths (say greater than 45 MPa at 28 days), or when there are aggressive chloride or other chemical exposure conditions, CSA Type SF silica fume may also be used. Fly ash is typically used at between 15 to 20 percent by mass of cement and silica fume at between 5 to 8% by mass of cement. Other supplementary materials available in Western Canada, such as calcined metakaolin are now also starting to be used.

3.2 Aggregates

Most structural shotcrete in Western Canada is produced with either 14 or 10 mm maximum size coarse aggregate and a regular concrete sand. If 10 mm maximum size coarse aggregate is used, then the composite blend of coarse and fine aggregate is required to meet the ACI 506R-05 Grading No. 2 limits shown in Table 1 that follows.

Natural rounded fluvial or glacio-fluvial gravels and sands are readily available in Western Canada and make very good shotcrete aggregates. The coarse aggregate fractions are typically partially crushed. Coarse and fine aggregates are required to

Table 1. Aggregate grading.

Metric sieve size	Total passing each sieve (% by mass)
14 mm	100
10 mm	90–100
5 mm	70–85
2.5 mm	50–70
1.25 mm	35–55
630 µm	20–35
315 µm	8–20
160 µm	2–10

meet the limits for physical properties and deleterious substances detailed in CSA A23.1-04, Table 12.

3.3 Chemical admixtures

Water reducers and or high range water reducing admixtures conforming to the requirements of ASTM C1141 are used in nearly all structural shotcrete. They reduce the water demand of the shotcrete for pumping and shooting. This enhances the *shootability* of the mix, that is, increases the height to which the shotcrete can be applied without sagging and sloughing (fall-out) when bench gunned to the full thickness of the element in a single pass. The reduction in water demand also helps to reduce the shrinkage capacity of the shotcrete and potential for restrained shrinkage cracking.

Air entrainment, as previously mentioned, is required to provide durability in freezing and thawing exposure environments. Contractors have, however, found it beneficial to add an ASTM C260 air entraining admixture to the shotcrete to enhance *pumpability* and *shootability*. As the air content is increased, the slump of the shotcrete goes up, making it easier to pump. However, on impacting on the receiving surface about half the air content is lost and there is an instantaneous stiffening of the in-place shotcrete. This enhances the thickness and height of shotcrete build-up achievable without sagging and sloughing. This process is well described by Jolin & Beaupré (2000) as the so-called *slump killing effect*.

Shotcrete is usually ordered in load sizes such that it can be discharged from a ready-mix concrete truck within a suitable period of time that is not necessary to use retarding or hydration controlling admixtures. Supply of 6–8 m³ size loads is common. Re-tempering with water should be avoided, as it can be detrimental to placing and finishing characteristics and increase the shrinkage and hence cracking potential of the shotcrete.

4 FORMWORK AND REINFORCING

4.1 Formwork

Much of the economic benefit arising from the use of the shotcrete process, in lieu of conventional cast-in-place formed concrete construction arises from either eliminating formwork (as in below grade shooting directly against the waterproofing system installed on excavated earth or rock, or temporary stressed tie-back shotcrete supported excavations) or use of single sided forms.

When single sided forms are used they should be sufficiently robust and braced and secured such that they can support the weight of the steel



Figure 2. Single sided formwork for shotcrete walls.

being erected (and men installing the steel), applied shotcrete and wind loads without overturning. Figure 2 shows one type of single sided formwork.

While the force of the shotcrete on the formwork during application is small, Gagnon & Jolin (2007), and there is hardly any pressure on the formwork from the freshly applied shotcrete (it is essentially self supporting), the shotcrete should be allowed to develop sufficient strength before formwork removal. This particularly applies to high walls, in order to be able to resist wind loads and any forces from stripping the forms.

In elements such as pilasters, columns or exterior corners of intersecting walls, the use of shooting wires, instead of edge forms should be considered, as they are more conducive to the shooting process than external edge forms.

4.2 Reinforcement

The same reinforcing steel required by the structural engineer for formed and poured cast-in-place concrete construction is typically also used in reinforced shotcrete construction. Many basement walls require two layers of 20 M or 25 M reinforcing steel vertically and horizontally with 15 M shear reinforcement. Wall thickness can vary from 450 mm at the base of a 5 floor deep underground parking structure to 250 mm thick at the top floor. Such reinforcing can be readily encased by a competent shotcrete nozzleman using the bench gun shooting process. Figure 3 shows a wall mock-up for a project in Calgary, Alberta. Figure 4 shows shooting of the mock-up and Figure 5 shows finishing of the mock-up.

Much more complex structural elements such as pilasters or shear walls with three or even four layers of reinforcement can also be shot in a single pass using the bench gun shooting process, but this requires a special level of skill and technique on the



Figure 3. Mock-up of structural wall for Calgary parking structure.



Figure 4. Shooting a mock-up.



Figure 5. Finishing of the mock-up.

part of the nozzleman and shotcrete crew, as will be demonstrated in a case history example which follows.

Having said the above, an important consideration in structural shotcrete construction is the reinforcing steel detailing. The structural design engineer should work with the contractor at the design stage, with input from the shotcrete contractor, regarding how to best fix the steel to make it readily conducive to encapsulation during the shooting process. ACI 506R-05 *Guide to Shotcrete* provides guidance but here are some general recommendations:

- Avoid the use of multiple lap splices
- Chair all rebar off the substrate (earth or rock face) or back form such that the shotcrete can build from behind to provide the required cover and fully encapsulate the bars
- With multiple layers of rebar, ensure the front rebars are spaced such that the nozzle can be thrust between them to facilitate encapsulation of the back layer of rebars.
- Firmly secure vertical and horizontal rebars at sufficient locations with tie wires that the bars do not vibrate or move during shooting; avoid large knots of tie-wire which could create shadows and voids during shooting.

5 SHOTCRETE MIXTURE DESIGN AND PERFORMANCE REQUIREMENTS

A typical performance specification for a structural shotcrete wall is given Table 2 below. An example of a shotcrete mixture design for structural walls for an in-the-ground five floor deep parking structure which satisfied the above performance requirements is given in Table 3.

Table 2. Performance requirements.

Test description	Test method	Age (D)	Specified requirement
Maximum W/B			0.45
Air content—As shot ¹	CSA A23.2-4C	—	4 ± 1%
Slump at discharge into pump	CSA A23.2-5C	—	60 ± 20 mm
Minimum compressive strength, MPa	CSA A23.2-14C	7 28	30 MPa ² 40 MPa ²

Notes: 1. To obtain an *as-shot* air content of 4 ± 1% will require an air content at the point of discharge into the shotcrete pump in the 7 to 10% range.

2. The Engineer may specify a higher design compressive strength if structurally required.

Table 3. Typical shotcrete mixture design.

Material	SSD density Agg [kg]	Density [kg/m ³]	Volume [m ³]
Cement type GU	400	3150	0.1270
Flyash type F or CI	80	2070	0.0386
Coarse aggregate (14–5 mm, SSD)	400	2670	0.1498
Concrete sand (SSD)	1230	2620	0.4695
Estimated water, L	185	1000	0.1850
Water reducing admixture, L	1.200	1000	0.0012
HRWR*, L	*	1000	*
AEA**, L (for 7–10% as batched)	**	1000	**
Air content $\pm 1\%$ (as-shot)	4.0	—	0.0405
Total	2266	Yield (m ³)	1.0016

Notes: * Add high range water reducing admixture at appropriate dosage if required to achieve the maximum allowable W/CM ratio and required slump.

** Adjust air entraining admixture dosage as required to meet air content 7–10% as batched and 3–5% as-shot.

6 SHOTCRETE SUPPLY AND APPLICATION

6.1 Shotcrete supply

Most wet-mix shotcrete for structural shotcrete applications has been by ready-mix supply with either central mix or transit mix batching and mixing. Volumetric site batching and dry-bagged premix supply is only occasionally used. Most shotcrete application is done with piston pumps custom designed for use with shotcrete. A number of equipment manufacturers have designed piston pumps with swing tubes specifically for use in shotcrete construction, although the pumps can also be used for pumping mortars and up to 20 mm maximum size aggregate conventional concrete. Peristaltic pumps and ball seat type pumps are occasionally used but are not really recommended as they do not provide the same shotcrete impacting force to wrap shotcrete around rebar and productivity of a good wet-mix shotcrete swing tube piston pump.

6.2 Shotcrete application

Most structural shotcrete walls are constructed using hand nozzling and the *bench-gun* shooting method. This process is diagrammatically illustrated in Figure 6 above.

Shooting wires are used to control the cover to reinforcing steel and final line and grade. Some contractors will shoot the structural shotcrete wall

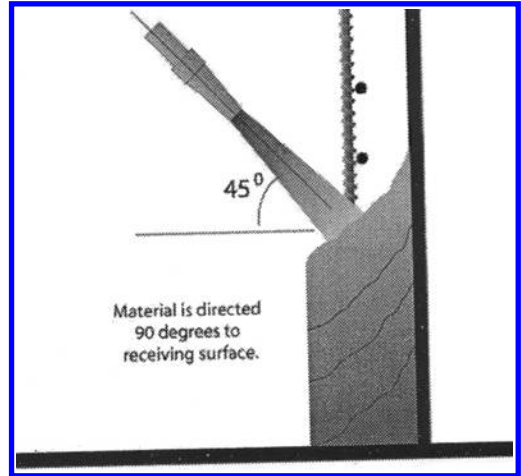


Figure 6. Bench gun shooting method.

to the final thickness in one pass. Others prefer to bench gun shoot so as to just cover the outer layer of rebar, trim off any high spots or excessive irregularities with cutting rods and/or trowels and then allow the shotcrete to stiffen, before shooting a final finish coat vertically. The advantages of the latter method are:

- The chances of finishing operations causing sags or tears in the bench gunned shotcrete are virtually eliminated.
- It is easier for the finishers to cut, trim and finish the freshly applied approximately 40 mm thick finish coat, then the aging and stiffening bench-gunned shotcrete.

7 SHOTCRETE FINISHING AND CURING

7.1 Shotcrete finishing

After trimming the shotcrete to line and grade with a cutting rod, the surface is closed and smoothed with finishing devices such as darbys, hand floats and trowels. After the shotcrete has stiffened sufficiently, the final surface finish is applied. Some architects specify very smooth steel trowelled surface finishes, but the preferred finish (particularly for surfaces which are to receive a coating or paint) is a textured surface finish made using either a magnesium float (light texture) or sponge float (intermediate texture) or wood float (coarser texture).

A well finished structural shotcrete wall should be more uniform and pleasing in appearance than a cast-in-place structural concrete wall; there are no form board joint marks, form tie holes, pour lines, honeycomb or segregation and grinding to remove

bins and sacking and rubbing are not required. This provides cost benefits for the contractor and aesthetic benefits for the architect and Owner.

7.2 Curing

It is important to thoroughly cure the freshly applied and finished shotcrete to prevent plastic shrinkage cracking and crazing, particularly if hot dry and/or windy evaporation conditions prevail. This can be done using techniques such as sun-shades/protection barriers, fogging sprays, or spray-on curing compounds. Any curing compounds used should be compatible with any subsequent coatings or paint.

To minimize the occurrence of restrained drying shrinkage cracking, the shotcrete is best moist cured using techniques such as saturated burlap (hessian) covered with plastic sheets, or preferably non-staining plastic coated non-woven synthetic fabric curing blankets, kept wet for at least 4 days and preferably 7 days if the construction schedule permits. Alternatively, spray on curing compounds are sometimes used where moist curing is not practical.

In cold weather conditions the use of hoarding and heating and/or thermal curing blankets may be required. In some of the Calgary, Alberta projects shotcrete work progressed right through the winter with ambient temperatures falling to as low as—30°C. Careful attention to hoarding, heating (using vented heaters that do not cause carbonation) and curing are required in such extreme conditions.

8 CASE HISTORY EXAMPLES

8.1 Calgary parking structure

In 2007–2008 the exterior perimeter walls of a city-block size five floor deep in-the-ground parking structure for a high rise building in downtown Calgary, Alberta, was constructed using the wet-mix shotcrete process, in lieu of conventional formed and cast-in-place concrete construction. The excavation was supported by steel soldier piles and timber lagging, as shown in Figure 7.

A bentonite filled synthetic waterproofing membrane was installed against the soldier piles and lagging and the double layers of vertical and horizontal rebar fixed. Figure 8 shows the steel fixed, ready for shooting.

All footing/wall intersections are filled first (coved) to prevent entrapment of rebound and overspray at such locations. Bench gun shooting then proceeds on a diagonal up to the full height of one floor (about 3 m), as shown in Figure 9.

Once the bench gunned shotcrete has stiffened sufficiently, the final coat (about 40 mm thick) is

vertically applied, starting at the top half of the wall. The top half is trimmed with cutting rods to the shooting wires and finished to line and grade. This is then followed by shooting the lower half of the wall, as shown in Figure 10. Figure 11 shows trimming the lower half of the wall with a cutting rod and Figure 12 shows the final finished wall. Figure 13 shows the site with four levels of the structural shotcrete walls completed.



Figure 7. Five floor deep parking structure excavation with steel soldier piles and timber lagging.



Figure 8. Two layers of steel fixed, ready for shooting.



Figure 9. Bench gun shooting process.



Figure 10. Shooting final finish coat in lower half of wall.



Figure 13. Parking structure with four levels completed.



Figure 11. Trimming shotcrete to shooting wires with cutting rod.



Figure 14. Stripped back face of shotcrete applied to a single sided form.

Interior *crash walls* and some perimeter walls used single sided formwork and structural shotcrete. Figure 14 shows the stripped back side of one such perimeter wall. Note the dense quality of the shotcrete free of any shadows, voids or defects. This in-the-ground parking structure took about 10 months to complete and included shotcreting during a very cold winter, when temperatures fell to as low as -30°C .

This case history example represents the first use of this construction methodology in Calgary. An indication of the success of the structural shotcrete process is that the structural design engineers for this project have now also specified this method for a number of other large projects in Calgary and elsewhere in Western Canada, and the general contractor who did the work is now using this same construction method on a number of other projects. Other major contractors in Western Canada have also started using the same system and a strong group of in-house or specialty shotcrete subcontractors, with ACI Certified Nozzleman and experienced supervisors and crews are now performing structural shotcrete work in Western Canada.



Figure 12. Final finished section of wall.

8.2 Vancouver hotel

In 2008 construction started on a 50 storey high rise hotel in downtown Vancouver. The design included a requirement for very heavily reinforced seismic walls and pilasters in the several floors of underground parking. Normally such structural elements would be constructed using formed cast-in-place self consolidating concrete (SCC). The structural design engineer, however, had previously had successful experience in southern California in using shotcrete to construct heavily seismically reinforced structural concrete elements and asked a specialty shotcrete contractor in Vancouver if he could construct the walls and pilasters of interest using the wet-mix shotcrete process.



Figure 15. Reinforcing steel erected in a preconstruction wall and pilaster mock-up.



Figure 16. Bench gun shooting mock-up wall element.



Figure 17. Bench gun shooting pilaster mock-up.

The shotcrete contractor considered that he could and so a mock-up was constructed to assess the viability of using the shotcrete process for these elements on this project. Figure 15 shows a preconstruction mock-up of a wall and pilaster element. Figure 16 shows bench gun shooting a wall element and Figure 17 shows shooting a pilaster.



Figure 18. Judicial insertion of pencil vibrator to aid in shotcrete consolidation and wrap around rebar.



Figure 19. Diamond saw-cut section through wall and pilaster mock-up.

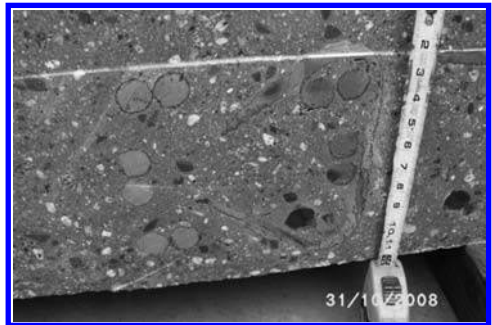


Figure 20. Close-up view of saw-cut section; note excellent encapsulation of reinforcing steel.

Figure 19 shows a cut section through the stripped wall and pilaster mock-up. Figure 20 shows a close-up view of a diamond saw cut section and Figure 21 shows a few of the many cores drilled through



Figure 21. Example of cores drilled through pilaster mock-up; note dense consolidation of shotcrete around rebar.

On the strength of this mock-up and the shotcrete meeting all the other performance requirements for the project, the shotcrete contractor was given the go-ahead to construct the below grade parking structure walls and pilasters for this prestigious 50 storey hotel building using the wet-mix shotcrete process. This work has now been successfully completed.

8.3 Whistler 2010 Winter Olympics bobsleigh/luge track

The 1.7 km long bobsleigh/luge track constructed at Whistler for the 2010 Winter Olympics was built using the wet-mix shotcrete process. The track is a complex curved structure with several layers of reinforcing steel, large diameter cooling pipes (30 mm dia) and very exacting finishing tolerances. A typical cross-section of a high-bank section of the track is shown in Figure 22.

Figure 23 shows installation of the cooling pipes on a section of the track. Figure 24 shows a section of the track constructed and ready for shooting.

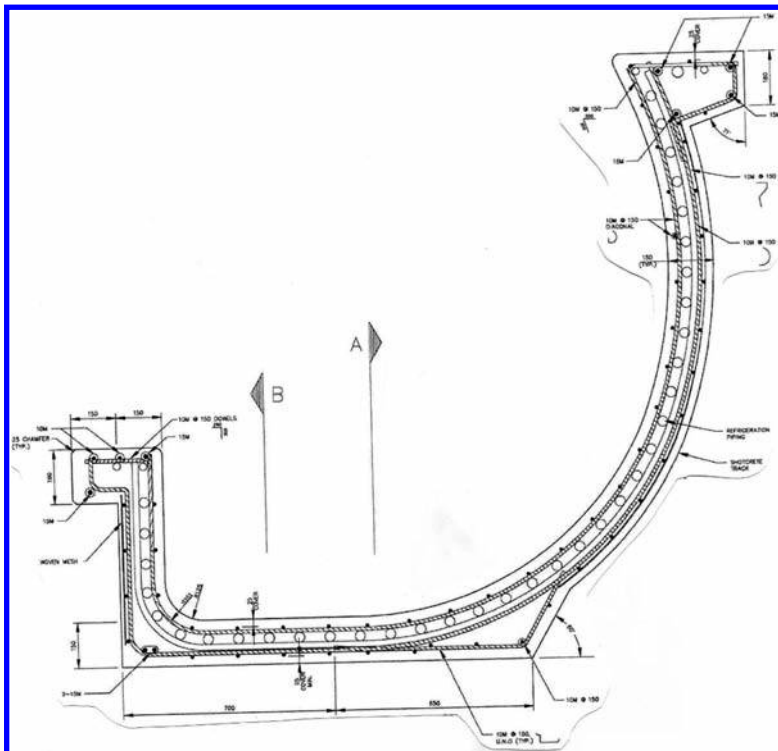


Figure 22. Typical cross-section of the Whistler bobsleigh/luge track (original design).

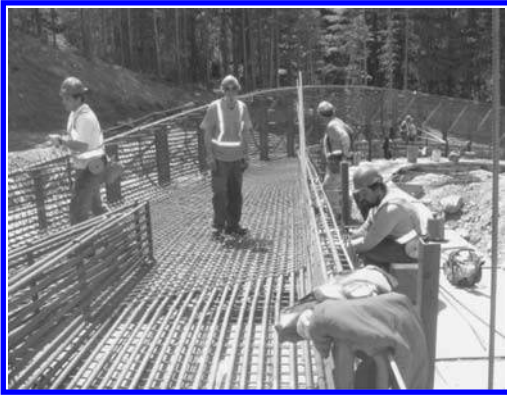


Figure 23. Installation of cooling pipes on a track section.

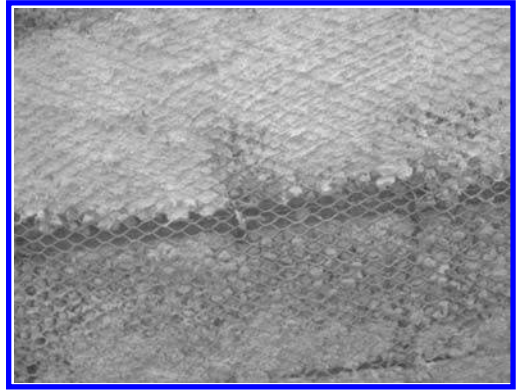


Figure 25. Back face of expanded lath formwork; note voids behind rebar and trapped coarse aggregate.

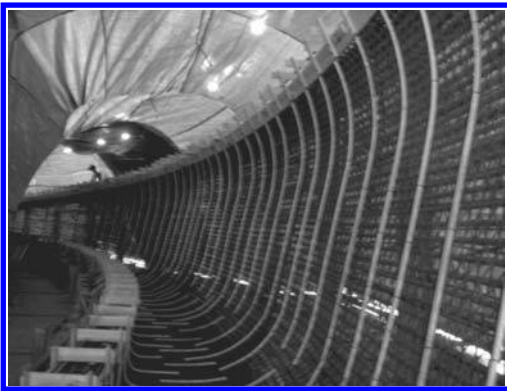


Figure 24. Section of a track with all hardware installed, ready for shooting (white plastic pipes are for screed control and are temporary).



Figure 26. Typical back face of track section after high pressure water blasting.

The white pipes are temporary screed pipes, placed to 3 mm geometric tolerance, to which the shotcrete is finished. They are removed as part of the finishing process.

The initial design called for the use of expanded metal lath to create the back form against which the shotcrete was placed. This proved to be problematic as the lath was fixed tight against the back layer of rebar and this did not allow for the shotcrete to envelope the rebar from behind and fully encapsulate it. Also, the lath openings were larger than desirable, allowing mortar to blow through the lath, as shown in Figure 25.

The consequence of this was that considerable effort had to be expended high pressure water blasting the back face of the assemblage to remove porous shotcrete and open up the shadows/voids behind rebar for subsequent shooting with the back lift of shotcrete. Figure 26 shows a typical

back face of track section after high pressure water blasting.

The contractor requested a modification to the steel installation and forming system. The position of vertical and horizontal steel was reversed and *Stay Form* ribbed metal lath, which self-chaired off the vertical back rebar was proposed. The engineer agreed to this modification, subject to the contractor, demonstrating in a mock-up that a quality structural shotcrete construction could be produced. Figures 27 to 32 shows construction of this mock-up. Figure 28 shows the redesigned mock-up ready for shooting. Note the use of *Stay Form*.

The wall/floor intersection was shot first (Figure 28) followed by the floor. The walls were shot (Figure 29). Shotcrete was pumped into temporary wood forms and vibrated to construct the bond beams at the head of the walls. The forms were then stripped (Figure 30) before application of the final finish coat (Figure 31).



Figure 27. Redesigned mock-up ready for shooting.



Figure 30. Stripped forms in bond beam, ready for shooting finish coat to track.



Figure 28. Shooting mock-up wall/floor intersection.



Figure 31. Application of final finish coat to track.

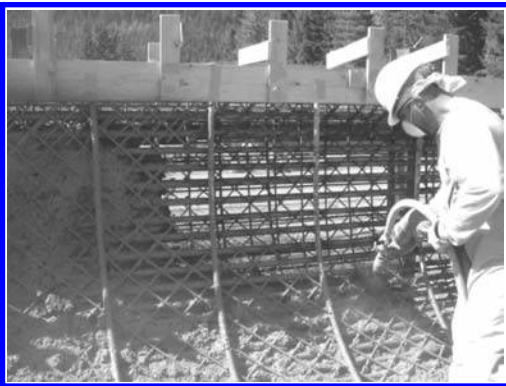


Figure 29. Shooting first lift on wall.



Figure 32. Texturing shotcrete after final finishing.

Figure 32 shows texturing the shotcrete after final finishing.

The mock-up section was then diamond saw-cut to examine the quality of encapsulation of shotcrete around reinforcing steel, cooling

pipes and *Stay Form*. Figure 33 shows one such cut section.

The quality of shotcrete encasement of all these elements was excellent and the work proceeded using this revised design and construction method



Figure 33. Diamond saw-cut section through mock-up; note good encapsulation of rebar and cooling pipes and good consolidation at Stay Form.



Figure 34. Section of completed bobsleigh/luge track.

with good results. Figure 34 shows a section of what has proven to be the fastest bobsleigh/luge track in the world.

9 CONCLUSIONS

Wet mix shotcrete has a long history of successful use for construction of reinforced structural walls, pilasters and other elements in California and Washington states in the USA. The past five years have seen the transfer of this technology to Western Canada. This paper provides an overview of important requirements for this technology transfer to take place and provides case history examples of the successful use of wet-mix structural shotcrete.

Two of the case history examples are for projects which would normally have been constructed with the conventional formed and cast-in-place concrete construction process, but where the use of shotcrete provided a quality project with savings in time and cost. The third case history example, the Whistler bobsleigh/luge track, because of the complex curved geometry of the track and very exacting finish tolerances is an example of a project which could only be reasonably constructed using the shotcrete process.

With the aid of the American Shotcrete Association (ASA) education programs American Concrete Institute (ACI) shotcrete nozzleman certification program, and the development of more engineers knowledgeable in the shotcrete process, and more qualified and experienced shotcrete contractors, superintendents and nozzlemen, it is expected that the next decade will see increasing use of the wet-mix shotcrete process for structural applications in Western Canada and elsewhere in North America.

ACKNOWLEDGMENTS

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REFERENCES

- ACI 506R-05 Guide to Shotcrete.
- ASTM C1140 Standard Practice for Preparing and Testing Specimens from Shotcrete Panels.
- CSA A3000-03 Cementitious Materials for Use in Concrete.
- CSA A23.1-04 Concrete Materials and Methods of Concrete Construction.
- Gagnon, F. & Jolin, M., 2007, Dynamic Forces During Shotcreting Operations, *Shotcrete Magazine*, Vol. 9, No. 3, Summer 2007, pp. 26–28.
- Jolin, M. & Beaupré, D., 2000, Temporary High Initial Air Content Wet Process Shotcrete, *Shotcrete Magazine*, Vol. 2, No.1, February 2000, pp. 22–23.
- Morgan, D.R. & Totten, L., 2009, Structural Shotcrete Specifications, *Shotcrete for Africa* Conference, The Southern African Institute of Mining and Metallurgy, Misty Hills, South Africa, 2–3 March 2009, pp. 23–40.

Centrifugal placed concrete for lining horizontal pipes, culverts, and vertical shafts

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ABSTRACT: Conventional dry-mix shotcrete (where water is added at the nozzle) or wet-mix shotcrete (where air is added at the nozzle to pneumatically convey the material to the receiving surface) has long been used for lining horizontal pipes, culverts, and vertical shafts. This paper, however, describes a different technology for placing concrete on such surfaces: the use of Centrifugal Placed Concrete (CPC). Instead of adding water at the nozzle (as in dry-mix shotcrete), or air at the nozzle (as in wet-mix shotcrete), the concrete mix is pumped to a spinning head, which spins at between about 4000 to 5000 rpm. The concrete is then centrifugally projected onto the receiving surface at high impacting velocity. This paper describes the use of this CPC technology for lining horizontal pipes, culverts and other circular structures and vertical raise-bore shafts. A case history example is described for lining of a vertical raise-bore shaft at New Gold Inc's New Afton copper-gold mine near Kamloops in British Columbia, Canada, using a macro-synthetic fibre reinforced concrete. This is believed to be the first use of this technology in North America, if not the world. Productivity with the CPC was much greater than that previously achieved using a conventional robotically applied dry-mix, steel fibre reinforced shotcrete to line similar vertical shafts at the mine.

1 INTRODUCTION

Since 1991 Shotcrete Technologies Inc. has provided remotely-controlled equipment and know-how for wet-mix shotcrete lining of many horizontal pipes, culverts, and vertical shafts in tunnels and mines. Some of these horizontal structures were over 1000 m long and some of the shafts were over 100 m deep. Pipe and culvert diameters ranged from as little as 450 mm to as much as 2.5 m. Shafts were typically in the 2.5 to 3.5 m diameter range. Plain and synthetic or steel fibre reinforced shotcrete mixes have been used in many such applications, depending on the engineer's requirements. A remotely-controlled wet-mix shotcrete lining rig is shown being winched through a pipe in Figure 1. Visual monitoring is provided by a television camera attached to the robotic rig. Figure 2 shows a similar wet-mix shotcrete robotic rig being used to line a raise-bore shaft at a mine.

While the preceding systems have generally worked well and provided high quality linings,



Figure 1. Robotic lining of pipe with conventional wet-mix shotcrete.

they are not without their challenges. There are many moving parts in the robotic rig that need to be maintained and the shotcrete nozzle requires regular cleaning in order to maintain an optimal placement pattern. Recognizing this, Shotcrete

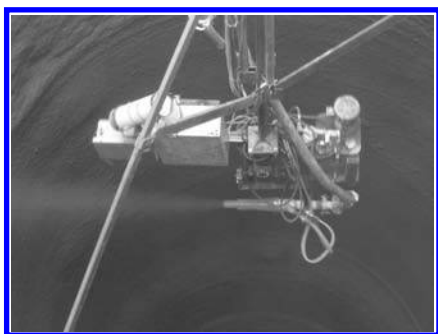


Figure 2. Robotic lining of raise bore shaft with conventional wet-mix shotcrete.

Technologies Inc. embarked on development of an innovative new system for projecting concrete; the development of the centrifugal placed concrete (CPC) spinner head system. A list is provided in Table 1 of some of the projects where robotically applied CPC has been used for lining horizontal pipes, culverts and other structures. Most of this paper, however, concentrates on the CPC lining of a 3.5 m diameter 290 m deep raise-bore shaft at New Gold Inc's New Afton copper-gold mine near Kamloops, British Columbia, Canada in June 2009, using a macro-synthetic fibre reinforced CPC.

Table 1. List of pipes and culverts lined with centrifugal sprayed concrete.

Date	Project	Location	Type	Dimensions	Thick	Remarks, all structural repair
2005	Union Pacific Railroad	Hotchkiss, Colorado	CMP	2500 If ×42" ID	¾"	Lined 400 lf/day for 7 days
2005	Mesa County, Colorado	Grand Junction, CO	CMP	110 If ×36" ID	1"	Lined in one day
2005	Elbert County, Colorado	Simla, Colorado	CMP	85 If ×96" ID	1"	2 Culverts at this dimension
2005	City of Glenwood Springs	Glenwood Springs, Colorado	CMP		½"	Several culverts, multiple diameters
2005	Town of Castle Rock	Castle Rock, Colorado	CMP	450 If ×60" ID	1"	Storm sewer
2007	Kiewit Western	Rocky Mtn National Park	CMP	96 If ×24" ID	¾"	45 degree grade in national park
2007	El Paso County	El Paso, Texas	CMP	60 If ×60" ID	1"	2 Culverts at this dimension
2007				80 If ×36" ID	1"	3 Culverts at this dimension
2008	City of Idaho Springs	Idaho Springs, Colorado	Brick	Manhole		3 Manholes
2009	El Paso County	El Paso, Texas	CMP	60 If 36" ID	1"	6 Oval culverts, structural repair
2008	Jeffco Road & Bridge	Colorado	CMP	38 If ×18" ID	1"	3 Culverts on golf course
		Colorado	CMP	42 If ×18" ID	1"	Difficult access
2008	Mt. Snow Resort	Colorado	CMP	28 If ×24" ID	1"	
		Vermont	CMP	450 If ×48" ID	¾"	Reline culvert under parking lot
2008	Kansas DOT	Topeka, Kansas	CMP	120 If ×72" ID	2"	Dam outlet
				50 If ×30" ×50"	¾"	Under I-70 various oval
				55 If ×23" ×36"	¾"	Oval highway culvert
				196 If ×42" ×32"	¾"	Oval highway culvert
				175 If ×44" ID	¾"	Circular, stepped inclines
2009	BNSF Railroad (Aecom)	LaJunta, Colorado	Tile	550 LF ×5.5 ft	1½"	Storm sewer—diesel inflow
2006	MMWS Sewer District	Milwaukee, Wisconsin	Concrete	9000 If, 60" ID	¾"	Sewer tunnel & various manholes
2009	Sequoia National Park		CMP	Various	1"	9 Various lengths and diameters
2005–09	CO Dept of Transportation	Colorado	CMP	Various		Many culvert repairs under I-70

CMP = Corrugated Metal Pipe Culvert.

2 CENTRIFUGAL PLACED CONCRETE SYSTEM

The spinning head used on the gold mine project is shown in Figure 3. The small hose is a 25 mm diameter line connected to the coupling which feeds compressed air into the spinner head to rotate it at 4000 to 5000 rpm. The greater the volume of air provided, the faster the head spins. The concrete is pumped into the spinner head, which has a conical dispersion feature.

As the head spins the pumped concrete is centrifugally projected at high impacting velocity onto the receiving surface (pipe, culvert or shaft). The ejected material dispersion and velocity is such that unlike conventional wet-mix shotcrete, where a dense shotcrete stream ejecting from the nozzle is clearly visible, the ejected material is barely visible. One simply sees material building up on the receiving surface. The rate at which the material builds up on the receiving surface depends on the diameter of the pipe, culvert or shaft and the rate at which the spinning head is pulled through the pipe, culvert, or shaft.

Start-up of production of an ASTM C1550 round panel in an above ground test chamber at the gold mine project is shown in Figure 4. Figure 5 shows the filled test panel prior to cutting back and trowelling to provide a flat surface. There is some rebound of concrete and fibre on the floor at the base of the test chamber. Systematic studies to quantify the amount and constitution of the rebound materials have not yet been conducted, but are planned prior to lining the next raise-bore shaft at the gold mine project. However, reports from the

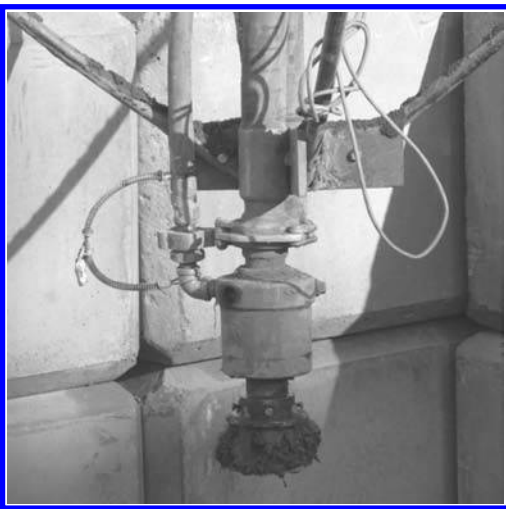


Figure 3. Spinner head assembly connected to pump hose and compressed air line.



Figure 4. Start-up of centrifugal placing of an ASTM C1550 round test panel in an above ground test chamber.

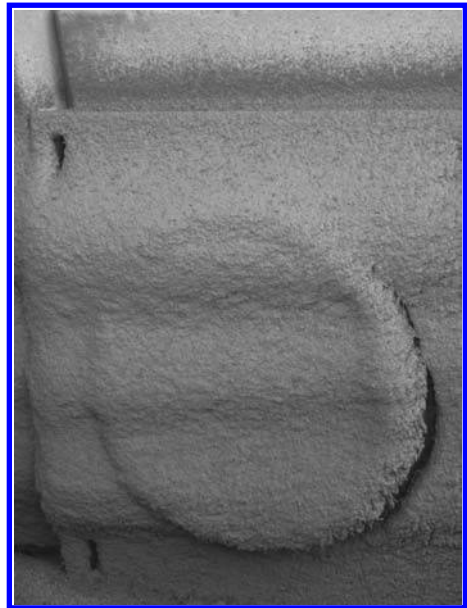


Figure 5. Sprayed ASTM C1550 Round panel prior to cutting back to flat surface.

mining engineers at the project on completion of lining the approximately 290 m deep, 3.5 m diameter raise-bore shaft lined in June 2009 indicated that very little rebound material was found at the base of the shaft.

At the New Afton mine the specified concrete lining thickness was a minimum of 50 mm. In producing the ASTM C1550 test panel in the above-ground test chamber the concrete was centrifugally applied to a thickness of 100 mm on the panel and test chamber walls, before cutting back the round test panel to the required 75 mm thick-

ness. Even at 100 mm concrete thickness, adherence to the test chamber walls was excellent and there was no sagging or sloughing of the concrete. This is for concrete which was discharged into the pump from the ready-mix concrete truck at about 170 mm slump. Note that no accelerator was used in this concrete. Special concrete mixture designs are required to enable this type of performance. Concrete mixture proportioning is discussed in the next section.

3 CENTRIFUGAL PLACED CONCRETE MIXTURE DESIGNS

Since 2005 Shotcrete Technologies Inc. has been using a proprietary natural-pozzolan-modified concrete, which also contains a superplasticizer, 'Kelcrete' rheology modifier and a low dosage of micro-synthetic fibre reinforcement for robotically lining horizontal pipes, culverts and other circular structures. Such centrifugal placed concrete has typically had a 5 mm maximum size aggregate, as lining thicknesses have mainly been in the 20 to 40 mm range. This mixture formulation has worked well in these horizontal lining applications, with almost no rebound and good adhesion and productivity. Review of cast concrete cylinder test results from concrete supplied to centrifugal placed concrete projects shows that this mixture design typically produces compressive strengths of around 40 MPa at 28 days.

For the gold mine project it was decided to evaluate the potential for use of a macro-synthetic fibre reinforced concrete for lining raise-bore shafts, using the CPC process. Proof-of-concept trials were conducted in Vancouver, British Columbia in December 2008. Two different concrete mixes were evaluated: a natural pozzolan mix provided by Shotcrete Technologies Inc.; and a silica fume modified mix designed by AMEC. Both mixtures contained 6 kg/m³ of Barchip Shogun macro-synthetic fibre reinforcement. The gold mine project requirements were as follows:

- Slump: 150 ± 20 mm (at end of hose),
- Maximum aggregate size: 5 mm (aggregate to meet ACI 506R-05 grading No. 1 requirement),
- Maximum water/binder ratio: 0.45,
- Minimum compressive strength: 30 MPa at 7 days, 40 MPa at 28 days,
- ASTM C642 at 28 days: maximum absorption: 8%, maximum volume of permeable voids: 17%,
- ASTM C1550 round panel test: target 320 J at 28 days at 40 mm central deflection.

The trials in Vancouver demonstrated that it was possible to place both the natural pozzolan mix and the silica fume mix containing 6 kg/m³

Table 2. Silica fume concrete trial mix design.

Material	Mass (kg)	Density (kg/m ³)	Volume (m ³)
Cement type GU	428	3150	0.1359
Silica fume	44	2200	0.0200
Fine aggregate (SSD)	1647	2670	0.6169
Water	199	1000	0.1990
Barchip Shogun fibre	6	900	0.0067
Superplasticizer	0.66	1000	0.0007
Air content (as shot)	(2.0%)	–	0.0200
Totals	2324	–	0.9992

of macro-synthetic fibres, using the CPC system. Mixture proportions for the silica fume mix are provided in Table 2.

Mixture proportions for the natural pozzolan mix are not provided in this paper, since for logistics and costs reasons, it was subsequently decided not to import the natural pozzolan from the USA to the New Afton project in British Columbia.

The silica fume mix produced compressive strengths (on cores extracted from CPC test panels) of 73 MPa at 8 days and a toughness at 7 days on a ASTM C1550 test panel of 389 J. Clearly, the CPC was of very high quality. It was, however, observed that the silica fume mix was very sticky and consequently did not project out as efficiently as the natural pozzolan mix (lower rate of productivity). In order to address this concern, a decision was made to trial a different, less sticky supplementary cementing material. The selected product was Whitemud Resources calcined metakaolin from Saskatchewan, Canada.

Hand cast laboratory trials were conducted at the AMEC Vancouver laboratory and subsequently CPC trials were undertaken at the gold mine project site using a metakaolin mix. While some CPC incorporating both metakaolin and a reduced amount of silica fume was also used in the trials and in some shaft lining work, most of the shaft lining was completed using the metakaolin mix detailed in Table 3.

Note that in this mix design the synthetic macro-fibre addition rate was reduced to 5 kg/m³. This was done as a precautionary measure, to reduce the potential for blockages during pumping down a 290 m deep shaft. This mix design generally placed well, provided the slump at the end of the hose was in the specified slump range of 150 ± 20 mm. A hydration controlling admixture was added to keep the pumped concrete live for an extended period of time, given that it had to be pumped through at least 300 m of hose. Figure 6 shows the hose laid out at the raise-bore shaft site. The hydration controlling admixture dosage rate was varied depending on the ambient

Table 3. Metakaolin concrete mix design.

Material	Mass (kg)	Density (kg/m ³)	Volume (m ³)
Cement type GU	420	3150	0.1333
Metakaolin (Whitemud resources)	85	2570	0.0331
Fine aggregate (SSD)	1630	2670	0.6116
Water	200	1000	0.2000
Barchip Shogun fibre	5.0	900	0.0056
HRWR admixture (BASF Glenium 7101)*, L	2.5	1000	0.0025
Hydration control admixture (Delvocrete), L	8.0	1000	0.0080
Air content (as shot), %	2.0	—	0.0202
Totals	2343	Yield = 1.0118	

* High range water reducing admixtures adjusted as necessary to provide required slump.



Figure 6. 300 m of concrete hose laid out at raise bore shaft.

and concrete temperatures and the depth at which the spinner head was in the raise-bore shaft. Concrete was kept live for up to as much as 8 hours. Note that the summer temperatures at the gold mine often exceeded 30°C. The black hoses baked in the sun and it was found necessary to supply the concrete at a slump of 180 to 200 mm at the point of discharge into the pump in order to meet the specified slump range of 150 ± 20 mm at the end of the hose.

4 HORIZONTAL PIPE AND CULVERT LINING

Shotcrete Technologies has been using remote control, robotic centrifugal placed concrete for



Figure 7. Robotically applied centrifugal concrete lining horizontal metal culvert.

lining pipes, culverts and other horizontal structures since 2005. Table 1 provides a list of some of these projects. As can be seen, pipe diameters ranged from 457 mm (18 in) to 2438 mm (96 in). Lengths of the pipes ranged from 8.5 m (28 ft) to 762 m (2500 ft). One concrete sewer tunnel lined was 2744 m (9000 ft) long.

A robotically applied CPC lining is shown being applied to a corrugated metal pipe culvert in Figure 7. The robot is winched through the culvert, with the rate of travel of the winch controlling the lining thickness. Obviously the robot travels more rapidly in smaller diameter pipes, to apply a given thickness of CPC lining. A television camera is mounted on the robot to provide real time monitoring of the application. In areas with voids or cavities in the original structure the rate of winching can be slowed down to provide local void filling and a smooth CPC finish profile.

5 CPC LINING OF RAISE BORE SHAFT AT NEW AFTON GOLD MINE, BRITISH COLUMBIA

An elliptical guidance device referred as the ‘egg’ is shown in Figure 8 about to be lowered into the shaft at the gold mine project. This elliptical ‘egg’ was constructed from tubular steel and is designed to keep the spinner head centered in the shaft at all times during concrete placement. Interestingly, the ‘egg’ was not needed to keep the spinner head stable. During spinning the spinner head does not move around during centrifugal application; it remains stable in one position even if it is simply dangling freely from the suspended hose without connection to the egg. The egg has slightly smaller lateral dimensions than the raise-bore shaft, so

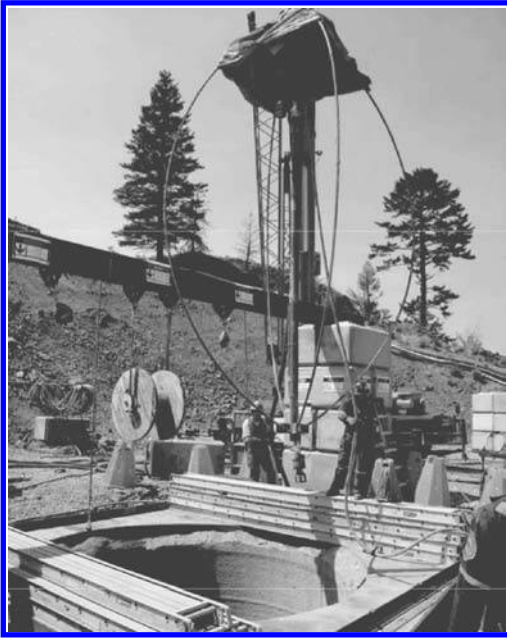


Figure 8. Top of raise bore shaft prior to centrifugal concrete lining application.

that it can be raised and lowered (even after lining spraying) without friction.

The shroud at the top of the egg protects TV cameras mounted on the central tubular pipe from damage due to any falling objects during placing. Figure 9 shows a close-up view of the cameras mounted on the egg while Figure 10 shows the general rigging set-up for the CPC lining of the raise-bore shaft. A 20 ton crane was used for lowering and raising the spinner head, egg, concrete pump lines, compressed air line and TV cable. The crane had a 22 mm diameter hoist cable to which the pump line was attached. Every 15 m (50 ft) of pump hose was connected with a custom-built coupling. Figure 13 shows this coupling. The U-bracket welded to the coupling was used in the rigging to connect the pump hose to the crane hoist cable.

A cable reel is shown in Figure 12 suspended from straps mounted on the steel beam spanning between the concrete lock-blocks shown in Figure 10, over which the concrete and air hoses pass as the egg is lowered down the shaft.

Two miners are shown in Figure 13 connecting the concrete pump line to the crane hoist cable. This was done using a 1.2 m long steel rope lanyard, connected with toggle bolts to the coupling shown in Figure 11, with two wire rope clips connecting the lanyard to the cable hoist. In this way the weight of the concrete pump hose (full of



Figure 9. View of TV cameras mounted at top of "egg".



Figure 10. General rigging set-up for CPC lining of raise bore shaft.



Figure 11. Coupling used for connecting lengths of pump hose and connecting pump hose to crane hoist cable.

concrete) was connected to the crane hoist rope every 15 m (50 ft). This had to be done for every 15 m (50 ft) of hose as the 'egg' was lowered down the shaft. The air line and TV cable were strapped



Figure 12. Cable reel over which the pump and air hoses pass.



Figure 13. Miners cinching bolts coupling wire rope lanyard to pump hose and crane hoist rope.

to the pump hose with duct tape. This enabled quick and easy disconnection of the air line and TV cable as the “egg” was raised.

As the egg was raised up the shaft during CPC lining operations, the bolts and lanyard had to be disconnected every 15 m (50 ft) so that the hose could be winched back over the cable reel. Proper rigging was key to the successful completion of this project and ergonomics and efficiency increased as

the work progressed. The miners wore full safety harnesses at all times while working on the platform and inside a 1 m stand-off distance from the edge of the shaft over the raise-bore shaft.

The logistics of lining the shaft varied somewhat, but the general concept was to pump the required length of 50 mm ID pump line for the day’s production full with concrete before attaching the egg and spinner head. The pump and spinner head was then started. When it was verified that the spinner head was operating correctly, the assemblage was then lowered down the shaft. The egg was lowered down fairly rapidly, placing about 6 mm of CPC onto the rock face in the shaft as it went down. Once the spinner head reached the lowest point for the day’s production, the egg was withdrawn slowly, so that the required full minimum 50 mm of thickness required for the shaft lining was produced. Ventilation in the shaft during CPC operations was kept neutral, or down-drafting, to enhance TV camera viewing and air quality for the workers.

A materials balance count was used to control lining thickness (i.e. a known volume of concrete discharged from the ready mix concrete trucks, applied over a known depth of shaft with an allowance for some materials loss through rebound). A special spring-loaded flow control device was installed on the pump hose 3 m above the spinner head to prevent free-fall of concrete onto the spinner head. A total of 198 m³ of concrete was used to line the shaft. Towards the end of the project between 35 to 40 m³ of CPC were applied to the shaft lining per working day. The work took 6 working days of actual shooting to line the shaft. Total time to complete the project was 10 days. This included time for orientation, rigging optimization and maintenance. The AMEC Kamloops laboratory carried out quality control testing on CPC. Cores extracted from test panels sprayed in a special above-ground test chamber produced 7 day compressive strengths averaging about 38 MPa and 35 day compressive strengths averaging about 70 MPa for the metakaolin modified mix detailed in Table 3; clearly a very high quality concrete.

Round panel tests to ASTM C1550 produced results of 270 J which was less than the target toughness value of 320 J. Increased fibre contents will likely be needed on future projects to meet the target toughness value of 320 J.

6 CONCLUSIONS

This paper demonstrates that centrifugal placed concrete (CPC) provides a viable alternative to conventional wet or dry-mix shotcrete for lining circular or slightly oval horizontal pipes and

culverts and circular vertical shafts. It is further demonstrated that CPC technology can be used with macro-synthetic fibre reinforcement to provide enhanced toughness to the concrete lining.

It has been demonstrated in this paper that CPC technology, with appropriate rigging, can be used to line a 3.5 m diameter by 290 m deep raise-bore shaft with high rates of productivity. CPC linings in such shafts can be completed in considerably less time than is achievable with robotically applied dry-mix, steel fibre reinforced shotcrete (10 days versus about 6 weeks). This is very attractive to the owner with regard to overall costs. Additionally, this paper demonstrates that with appropriate mixture designs and using conventional ready-mixed concrete batching, mixing and delivery, the CPC system is capable of producing high compressive strength and adequate toughness for most applications.

This paper presents a case history example of a North American first application of CPC with macro-synthetic fibre reinforcement for lining a large diameter (3.5 m) and deep (290 m) raise-bore shaft. It is expected that as similar shafts are lined

with CPC, further enhancements in rigging and placement methodologies will evolve, with further improvements in CPC quality and productivity and the economics of such linings.

ACKNOWLEDGEMENTS

The raise-bore shaft lining with centrifugal placed concrete at the New Gold Inc. gold mine near Kamloops, British Columbia, Canada was carried out as a joint effort between Shotcrete Technologies Inc. and New Gold Inc. Proof of concept placement in Vancouver, British Columbia, of macro-synthetic fibre reinforced CPC was provided by Bel Pacific Limited. AMEC Earth & Environmental, a division of AMEC Americas Limited, British Columbia provided CPC mixture designs and quality control testing during preconstruction trials and during shaft lining operations. Finally, the permission of New Gold Inc. to allow publication of this case history example of the raise-bore shaft lining with CPC is gratefully acknowledged.

Round and square panel tests—a comparative study

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ABSTRACT: In order to determine the Energy Absorption Capacity (EAC) of Fibre Reinforced Sprayed Concrete (FRSC) the practice in Norway has been to test round Ø600 mm panels on a continuous simple support. Today the method is challenged by the new European standard EN 14488-5 which prescribes square 600 mm panels, also with a continuous simple support. In this regard, a comparative study of round and square panels was undertaken by the Norwegian Sprayed Concrete Committee. The main scope was to carry out a comparative study of the EAC for the two methods. It was also a goal to gain experience regarding the practicalities associated with the two methods on-site, on handling of specimens as well as on details of the measuring procedures in the laboratory.

1 INTRODUCTION

In Norway the round panel method has been used for about a decade for determination of fibre effect and for Quality Control in tunnel projects. The round panel method is described in the Norwegian Concrete Associations Publication No. 7 (NB 7): “Sprayed concrete for rock support”. Today, however, this method is challenged by the square panel method, described in EN 14488-5 “Testing sprayed concrete—Part 5: Determination of energy absorption capacity of fibre reinforced slab specimens” and EN 14488-1 “Testing sprayed concrete—Part 1: Sampling fresh and hardened concrete”. Now it is quite evident, for both panel methods, that little is reported on aspects that appear to be important with regard to both sampling on-site and on details of the test procedure itself.

The present program of research was undertaken in order to collect documentation on energy absorption capacity (EAC) from the two panel methods in parallel sets of nominally identical panels and perform testing in two laboratories. The intention was also to gain experience in spraying, sampling and handling the two panel types on-site according to their respective standards and gain experience with respect to the practicalities of the test procedures in the laboratory.

2 TEST PROGRAMME

2.1 Concrete mix

A total of three test series were undertaken. The concrete mix used in the various test series was a typical Norwegian mix for sprayed concrete. There

were only minor differences in the concrete mix among the test series. The concrete mix is shown in Table 1. For Series 1, the only one that involved sprayed concrete, set accelerator was used, while for Series 2 and 3 no set accelerator was used. The fibre types included were Dramix RC 65/35, a steel fibre with hooked ends, and Enduro 600, a crimped macro-synthetic polypropylene fibre (macro pp-fibres).

2.2 Test series

During the period January 2007–February 2008 three test series were carried out, each series consisting of parallel sets of square and round panels. In addition to fibre effect, the effect of other factors was investigated. These included: spraying and sampling according to standards, age at testing

Table 1. Concrete mix with variations.

Material	Quantity (kg/m ³)
CEM II/A-V 42.5R	460–475
Silica fume	23–24 (46 for w/(c + 2s) = 0.39)
Sand 0–8 mm	1487–1546
Fibre content	See Table 2
Superplasticizer	3.7–6.2
Retarder*	0.47–0.98
Pump aid**	2.3–2.5
Internal curing agent***	4.6
Air entraining agent	1.4–3.4
w/c + 2 s	0.39–0.42
Free water (L)	208–214

*Series 3 A and 3 B;

**Series 3 B;

***Series 1 and 2.

EAC, orientation of the panel during testing, support frame for round panels, and supplementary tests. Table 2 gives an overview of the three test series. A total of 80 panels have been tested, 40 for each geometry, and there were 12 comparative sets of round and square panels.

2.3 Production and curing

2.3.1 Series 1

The panels were sprayed with a robot on-site according to the respective standards. Hence, the round panels were sprayed in their final size, whilst the square panels were sprayed 1×1 m and later saw-cut to the final size of 600×600 mm. To be able to trim the square panels in acceptance with the tolerances

in the standard ($100_{+5/-0}$ mm) it was necessary to reduce the accelerator from nearly 30 L/m^3 (as used with the round panels) to about 20 L/m^3 .

After spraying the panels were covered with plastic and left for 5 days in the tunnel. The round panels were placed horizontally on the ground, while the square stood with the same angle to the wall as when they were sprayed, 20° off the vertical. Six of the panels, 3 round and 3 square were then transported to SINTEF Building and Infrastructures laboratory in Oslo, and kept in water until testing. The rest of the panels were transported to the Norwegian Public Roads Administration (NPRA) Central Laboratory in Oslo and kept in plastic for another 9 days, then placed in water until testing.

Table 2. Summary of the three test series.

Series	$m = w/(c + 2s)$ Fibre dosage	No of panels of each geometry	Age at EAC testing	Orientation of sprayed/cast surface	Support	Supplementary tests
Series 1* Robot sprayed	$m = 0.42\text{--}0.43$ ** 20 kg steel fibre	6 + 6	56 / 57 days	NPRA: Round: Down Square: Down SINTEF: Round: Down Square: Up	Round: Plywood of birch Square: Steel + strips of a hard and smooth type of chipboard	Fibre content Compressive strength Density ***
Series 2 Trad. casting	$m = 0.41$ 20 kg steel fibre 35 kg steel fibre	5 + 5 5 + 5	49 days	Round: Down Square: Down	Round: Chipboard Square: Steel + strips of a hard and smooth type of chipboard	Slump Air content Concrete temp. Fibre content Compressive strength Density
Series 3 A Trad. Casting	$m = 0.40$ 15 kg steel fibre 30 kg steel fibre 45 kg steel fibre $m = 0.39$ 30 kg steel fibre	3 + 3 3 + 3 3 + 3 3 + 3	34 / 35 days	Round: Up Square: Up	Round: Plywood of birch Square: Steel + strips of a hard and smooth type of chipboard	Slump Air content Concrete temp. Air temp. Fibre content Compressive strength Density
Series 3 B Trad. casting	$m = 0.41$ 3.0 kg macro pp-fibre 5.5 kg macro pp-fibre 8.0 kg macro pp-fibre $m = 0.39$ 5.5 kg macro pp-fibre	3 + 3 3 + 3 3 + 3 3 + 3	37 / 38 days	Round: Up Square: Up	Round: Plywood of birch Square: Steel + strips of a hard and smooth type of chipboard	Slump Air content Concrete temp. Air temp. Fibre content Compressive strength Density

* 3 + 3 samples tested at two different laboratories;

** $m = 0.41$ before adding accelerator;

*** all tests on drilled cores, NPRA did not measure fibre content.

2.3.2 Series 2

When the second test series was to be carried out, it was decided to cast all the panels traditionally in their final size in order to delineate the panel geometry-effect on the EAC. The casting took place in a tent at site. Since the specimens were cast no set accelerator was used. After casting the panels were covered in plastic, and left at site lying horizontally on the ground for one day. They were then transported to the NPRA Central Laboratory and stored in water until testing.

2.3.3 Series 3

Casting and curing for the third series was the same as for the second series.

3 TEST METHODS

3.1 Supplementary tests

3.1.1 Slump

Slump of fresh concrete was measured according to the NPRA Handbook for laboratory testing (HB 014); this method is similar to the method described in EN 12350-2.

3.1.2 Air content

Fresh concrete air content was measured according to HB 014; a similar method is described in EN 12350-7.

3.1.3 Air- and concrete temperature

Air- and fresh concrete temperature was measured with an electronic thermometer.

3.1.4 Fibre content

For Series 1 fibre content was measured on drilled cores of hardened concrete, simply by using a magnet on the crushed concrete and subsequent washing and drying. One sample consisted of two specimens each with diameter 74 mm, and height 74 mm (Volume = 0.64 L). This is in accordance with the method described in NB 7 and EN 14488-7.

For Series 2 and 3 fibre content was measured by washing the fresh concrete. Steel fibres were gathered with a magnet, and synthetic plastic fibres were gathered with a landing net and then dried. In Series 2 the size of the concrete samples was 2.0 litres (about 4.5 kg), and in Series 3 the size of the concrete samples was 3.0 litres. This is mainly in accordance with NB 7 and EN 14488-7, except for the size of the samples; NB 7 requires 10 kg (about 4.5 L) sample size, whilst EN 14488-7 requires samples of 1–2 kg (0.5–1.0 L).

3.1.5 Compressive strength

For Series 1 compressive strength was measured on drilled cores with various h/d ratio, $d = 74$ mm and $h = 74$ –81 mm. The results were therefore

converted to compressive cylinder strength for cylinders with h/d ratio of 2.0. This is according to HB 014; EN 12504-1 describes no conversion to h/d ratio 2.0. For Series 2 and 3 compressive strength was measured on cast cubes $100 \times 100 \times 100$ mm, according to HB 014 and EN 12390-3.

3.1.6 Density

Density was measured on the same samples as compressive strength, by weighing in air and immersed in water. This is according to HB 014 and EN 12390-7.

3.2 Energy absorption capacity

3.2.1 Test rig

The test set-ups are shown in Figures 1 and 2. For the round panels the support fixture was made of plywood of birch (Series 1 and 3) or chipboard (Series 2). The plywood/chipboard support was 40 mm high and 50 mm wide and had an inner diameter of 500 mm. For the square panels the support fixture was made of steel, 20 mm thick and 500 mm internal dimension. Strips of a hard and smooth type of chipboard were used as bedding material. According to the EN-standard the bedding

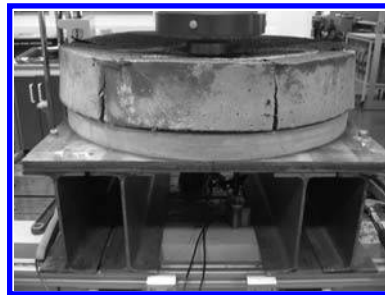


Figure 1. Set-up for round panels. The support fixture was made of plywood of birch/chipboard, no bedding material

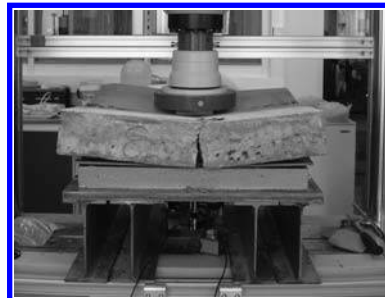


Figure 2. Set-up for square panels. Support of steel, strips of plywood as bedding material.



Figure 3. Placing of the two displacement transducers with metal caps.

material should be made of mortar or plaster, but this was considered too cumbersome and time-consuming to implement. The central displacement of the panels was measured by two displacement transducers as shown in Figure 3. The transducers are spring-loaded and have a measuring range of 50 mm. On top of the two transducers there were placed caps of metal to ensure that the transducers did not penetrate the cracks when they opened during testing.

A steel plate was put between the central load cell and the specimens, a $\varnothing 100$ mm cylindrical plate for the round panels (and a thin sheet of cardboard) and a 100×100 mm square plate (and a thin sheet of cardboard) for the square panels. The test machine had a maximum load capacity of 200 kN. The deformation rate during the test was controlled by the average signal from the two displacement transducers. Prior to the test, the load-cell was stabilized at a load of 1 kN. With this initial load the test was started.

The stiffness of the test machine (frame, load cell and loading block) is 235 kN/mm. With a support fixture made of plywood/chipboard, the total stiffness is unknown. With a support fixture made of steel the stiffness of the total load system satisfies the requirements in EN 14488-5.

3.2.2 Test procedure

Prior to testing, each panel was taken out of the water bath and transported to the test rig where the test was started within 45 minutes. The procedure was then as follows. The mid-point was marked on the side of the panel facing down during testing and then the panel was placed in the test rig and centred. Two displacement transducers were placed under the centre of the panel. The average of the two transducers forms the signal for deflection control. On the upper side of the panel a load plate (and a thin sheet of cardboard) was placed at the centre. The load cell is prepared for testing by lowering it to the load plate until a load of 1 kN is applied to the panel. The test was started and load and deflection signals were logged continuously by a computer. According to NB 7 the

load was applied deformation-controlled at a rate of 1.5 mm/min central deflection for the round panels, and according to EN 14488-5 at a rate of 1.0 mm/min central deflection for the square panels. The test was stopped automatically when the central deflection was 30 mm. The panel was lifted out of the test rig and the bottom side of the panel was photographed. The EAC was calculated as described in the standards as the area under the load-deflection curve from zero to 25 mm deflection. The results are corrected for thickness when deviating from 100 mm (see below).

3.3 Correction for panel thickness

The panel thickness influences the ability to absorb energy, where increased panel thickness will increase the energy absorption compared to the reference thickness. Consequently, the calculation of EAC should be corrected when the thickness deviates from the reference thickness. A theoretical evaluation of the effect of panel thickness has been reported (Thorenfeldt 2006). Target panel thickness is in this case $h_0 = 100$ mm. The following analysis procedure was proposed for panels with thickness h deviating from h_0 :

1. Absorbed energy should be calculated under the load-displacement curve between 0 and a modified displacement $\Delta_m = 25 \text{ mm} \times k$, where $k = 100/h$,
2. Calculated EAC should then be multiplied by the factor k ,
3. The final corrected EAC is then the result for the test.

The procedure assumes that four cracks develop and that the moment resistance at each crack is determined by the crack rotation angle. The total moment capacity is then linearly related to the thickness of the panel and the crack opening. It is likely that the correcting procedure will be valid within reasonable variations in panel thickness and that it will certainly contribute to achieving more comparable results. Note that neither NB 7 nor EN 14488-5 describes any procedures for correcting the EAC for deviations in panel thickness.

4 RESULTS

4.1 Supplementary tests

The results of the measurements done on fresh concrete are shown in Table 3, and Table 4 shows the results from the measurements done on hardened concrete. The high air content in some of the samples correlates with the lower compressive strength and density in the same samples.

Table 3. Measurements on wet concrete for all concrete mixes.

Series	Fibres	Slump (mm)	Air content (%)	Concrete temperature (°C)	Air temperature (°C)	Fibre content (kg/m ³)	
						Av.	Std
1 st Series	NPRA SINTEF*	20 kg steel fibre	–	–	–	–	–
		20 kg steel fibre	–	–	–	18.5	1.94
2nd Series		20 kg steel fibre	190	7	23.6	–	21.8
		35 kg steel fibre	210	6	22.5	–	35.6
3rd Series	w/(c + 2s) = 0.40	15 kg steel fibre	220	8.8	22	6	16.1
		30 kg steel fibre	240	12.5	19	6	37.0
		45 kg steel fibre	–	11.5	18	6	49.1
		3 kg synthetic fibre	180	8.5	19	10	3.3
	w/(c + 2s) = 0.39	5.5 kg synthetic fibre	180	8.5	19	10	7.8
		8 kg synthetic fibre	200	8.5	19	10	10.3
		30 kg steel fibre	–	11	18	6	32.7
		5.5 kg synthetic fibre	200	7.9	20	10	9.11

*Fibre content measured on drilled cores.

av. = average, std. = standard deviation.

Table 4. Measurements on hardened concrete for all concrete mixes.

			Compressive cube strength (MPa)				Density (kg/m ³)			
			7 days		28 days		7 days		28 days	
			av.	std.	av.	std.	av.	std.	av.	std.
1 st Series	NPRA* SINTEF**	20 kg steel fibre	–	–	56.6	6.62	–	–	2293	19.15
		20 kg steel fibre	–	–	60.8	3.16	–	–	2280	11.55
2nd Series		20 kg steel fibre	48.3	0.4	62.3	0.0	2250	0.0	2250	0.0
		35 kg steel fibre	42.5	0.0	60.0	2.1	2290	0.0	2295	7.1
3rd Series	w/(c + 2s) = 0.40	15 kg steel fibre	38.5	1.41	51.5	2.12	2185	7.07	2190	0.0
		30 kg steel fibre	29.5	0.0	39.8	0.35	2090	0.0	2100	0.0
		45 kg steel fibre	33.3	1.06	47.0	0.71	2130	0.0	2140	0.0
		3 kg synthetic fibre	43.3	0.35	57.8	0.35	2170	0.0	2180	0.0
	w/(c + 2s) = 0.39	5.5 kg synthetic fibre	49.0	0.0	64.0	2.12	2200	0.0	2210	0.0
		8 kg synthetic fibre	45.3	0.35	59.3	2.47	2190	0.0	2200	0.0
		30 kg steel fibre	37.0	0.0	51.8	1.06	2190	0.0	2200	14.14
		5.5 kg synthetic fibre	50.0	2.12	69.3	2.47	2225	7.07	2245	7.07

*Compressive strength measured on drilled cores, 56 days;

**Compressive strength measured on drilled cores, 28 days;

av. = average, std. = standard deviation.

4.2 Energy absorption capacity

The results from the EAC tests, including panel thickness and maximum load, are shown in Table 5. The correlation factors between square and round panels are also included in the table.

Average thickness for all panels was 102.0 mm, but 30% of the panels did not satisfy the requirements for panel thickness given in EN 14488-5 ($100_{+5/-0}$ mm). NB 7 does not give any tolerances for panel thickness. For the EAC the average coefficient of variation (COV) for round and square

Table 5. Panel thickness, maximum load, energy absorption capacity (EAC) for all concrete mixes (average, standard deviation and coefficient of variation in %), and the correlation factor for EAC between square and round panels. EAC are corrected for deviating panel thickness

			Panel thickness (mm)			Maximum load (kN)			EAC (J)			EAC square/round	
			av.	std.	COV	av.	std.	COV	av.	std.	COV		
Series		Fibres											
1 st Series	NPRA	20 kg steel fibre	R	101.9	2.37	2.33	50.6	1.09	2.15	727.1	70.31	9.67	
			S	101.8	3.82	3.75	52.8	2.01	3.80	527.0	73.66	13.98	0.72
2nd Series	SINTEF	20 kg steel fibre	R	103.3	1.53	1.48	57.0	4.56	8.00	717.4	83.38	11.62	
			S	99.5	0.71	0.71	63.7	3.18	5.00	583.6	98.09	16.81	0.81
		20 kg steel fibre	R	104.5	3.48	3.33	70.8	5.98	8.44	1143.5	37.42	3.27	
			S	104.9	1.49	1.43	79.8	8.15	10.22	1204.1	116.71	9.69	1.05
		35 kg steel fibre	R	104.4	1.84	1.77	93.1	5.13	5.51	1467.4	99.04	6.75	
			S	105.4	1.95	1.85	109.9	8.17	7.43	1645.3	103.30	6.28	1.12
3rd Series	w/(c + 2s) = 0.40	15 kg steel fibre	R	99.3	1.52	1.53	57.8	5.07	8.78	894.8	40.60	4.54	
			S	99.8	1.36	1.37	56.7	6.54	11.54	895.2	142.09	15.87	1.00
		30 kg steel fibre	R	101.1	2.39	2.36	43.5	3.54	8.15	686.9	53.87	7.84	
			S	101.2	1.55	1.53	41.6	5.06	12.16	561.3	77.93	13.88	0.82
		45 kg steel fibre	R	101.7	1.51	1.49	83.1	4.56	5.49	1487.6	39.11	2.63	
			S	101.4	1.24	1.22	75.0	5.72	7.63	1086.6	192.25	17.69	0.73
		3 kg macro pp-fibre	R	101.4	1.87	1.84	54.8	4.25	7.75	543.3	23.51	4.33	
			S	100.7	1.17	1.17	57.1	5.97	10.45	563.9	52.43	9.30	1.04
	w/(c + 2s) = 0.39	5.5 kg macro pp-fibre	R	102.2	2.76	2.71	65.5	1.62	2.48	935.9	107.49	11.49	
			S	101.2	1.73	1.71	65.9	6.94	10.53	896.9	28.41	3.17	0.96
		8 kg macro pp-fibre	R	102.7	1.73	1.68	68.9	7.58	11.01	1200.9	128.36	10.69	
			S	102.4	1.51	1.47	74.1	9.56	12.90	1226.6	233.00	19.00	1.02
		30 kg steel fibre	R	101.2	1.62	1.60	78.0	9.96	12.77	1399.3	208.39	14.89	
			S	101.3	1.38	1.36	66.2	3.08	4.65	978.0	70.39	7.20	0.70
		5.5 kg macro pp-fibre	R	103.9	3.46	3.33	66.0	2.05	3.11	998.0	38.82	3.89	
			S	101.2	1.05	1.04	59.5	8.89	14.95	912.6	144.41	15.82	0.91
Average correlation factor EAC, square/round												0.91	
Average COV, round												7.63	
Average COV, square												12.39	
Average COV, round and square												10.01	

av. = average, std. = standard deviation, COV = Coefficient of variation.

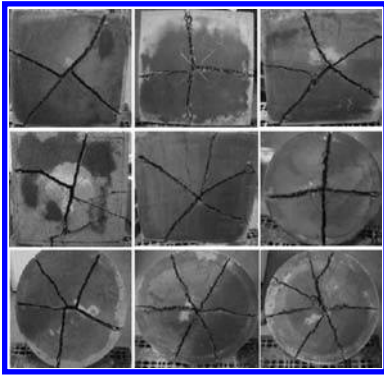


Figure 4. Different crack patterns; 4, 5 6 and 7 cracks appear, examples from different series.

panels for all sets in all series was 10%. Round panels displayed a somewhat lower COV (7.6%) than the square panels (12.4%). The average correlation factor between square and round panels for EAC for all sets was 0.91. Examples of different crack patterns are shown in Figure 4. Multiple cracking and shear crushing appeared for both round and square panels, and generally in panels with high fibre content.

5 DISCUSSION

5.1 Experiences from spraying panels

Prior to spraying the panels in Series 1, there were deliberations about how to spray the square panels. The surface area of the square panels is more than three times greater than the surface area of the round panels, and we were afraid that it would be too difficult to trim to the necessary thickness. These concerns were to a great extent confirmed during the trial.

The spraying of the round panels went as planned. The mould was placed on a steep angle to a rocky wall and sprayed. The mould was then carried away by two persons, placed horizontally on the ground and then trimmed by one person. This was possible to do with a normal accelerator dosage of 30 L/m³. When spraying the square panels, it soon became clear that the accelerator dosage had to be reduced; the first was damaged while trimming due to the extended time needed for the process and despite the fact that two persons were working hard. When the accelerator dosage was reduced to 20 L/m³, it was possible for two persons to trim the square panels, but it was very hard work. However, trimming the surface to an accuracy of $-0/+5$ mm was impossible; there were always some pits and bumps.

The round panels, with a weight of about 65 kg, could be handled by two persons. With a weight of about 230 kg, manual handling was impossible for

the square panels. When the square panels in addition have to be saw-cut to 600 × 600 mm, much larger machinery was required to be able to do the testing.

Another very important aspect of casting the square panels on-site is that the panels shall not be moved within 18 hours after being sprayed. This implies that the panels can not be produced where excavation is taking place and sprayed concrete is used. The close connection between in-situ use and testing will be lost, and the relevance of the testing will be reduced.

5.2 Experiences from testing energy absorption capacity

One of the observations during testing in the laboratory was that not all the panels were completely flat, especially the square panels. The round moulds were made of steel while the square moulds were made of wooden materials. The plywood used in the bottom of the square moulds was 5–6 mm thick, while EN 14488-1 requires at least 18 mm thick plywood or steel moulds. The wooden moulds were therefore probably not as stable with regard to weight and moisture, and the panels would easily be bent. Uneven panels mean that during the start of testing there will only be point contact between the panel and the support; hence the support is not continuous as required. This situation may influence the crack pattern, and the number of cracks. In particular the latter may affect the energy absorption during the test, and may have contributed to the increased scatter in results seen for square panels, see Table 5.

Another issue that might influence the crack pattern is if the sprayed surface is placed down during testing, and the surface is uneven with pits and bumps. This situation will also give rise to point contact with the support. In a tunnel lining of sprayed concrete there will be tensile stresses both at the inner side against the rock and on the outer exposed side. Hence, from this perspective the relevant orientation for panels in laboratory testing is unimportant. This may explain why the panel orientation is prescribed differently in the present standards (NB7 and EN). From a test methodology point of view, however, it appears to be sound to place the panels with the moulded/smooth surface against the support and apply the loading on the rougher trimmed/cast surface. This is related to the fact that the rougher surface will, if placed against the support, increase the chances for point contact with the support and may in this way influence the test results.

Panel thickness has been measured by different methods. Parallel measurements of thickness in a pattern over the entire surface and along the circumference gave quite similar average results. The former is a cumbersome way of doing it. In principle the thickness should be measured over the yield-lines,

after the test. Length/diameter was not measured, but a visually control when placing the panels in the test rig showed that some of the panels were just slightly oval/rectangular. Any possible effect of this is not investigated, but this effect is likely to be far smaller than the effect of panel thickness.

The panel thickness varied between 98.3 and 107.5 mm. Instead of discarding the panels with deviating thickness, the energy absorption was corrected for thickness. Even within the given tolerances, the EAC was noticeably over-estimated when not correcting for thicknesses deviating from 100 mm. An example from series 2 shows that a 105 mm thick panel gives, if not corrected for, up to 9% higher EAC when compared to a 100 mm thick panel. Hence, thickness should be corrected for panels even if they are within the given tolerances.

In Series 2 we made an important observation; the round chipboard support failed, see Figure 5. Occasionally the strips of hard and smooth chipboard used as bedding material for the square panels also failed. These two occurrences had to be a result of friction between the panel and the support, and made us wonder what effect friction had on the measured EAC. This aspect was investigated further in later tests, see Bjøntegaard (2009).

When testing of Series 3 A panels was carried out, there was a deviation in the deformation rate early in the test for some of the square panels with steel fibres. This deviation occurred for all the panels with $w/(c + 2s) = 0.40$ and 45 kg steel fibre and with $w/(c + 2s) = 0.39$ and 30 kg steel fibre, and also on one of the panels with $w/(c + 2s) = 0.40$ and 30 kg steel fibre. For the square panels that were tested with an early deviating deformation rate the following happened, for unclear reasons. The period with deviating deformation rate was from the start of testing and until around 3 seconds. The deflection was at this point in the range of 1 mm. With a correct deformation rate (1 mm/min) the deflec-



Figure 5. Failure in chipboard support.

tion should have been 0.05 mm after 3 seconds, hence the deviation was in the range of 20 times the normal load-rate. At this point (after around 3 seconds) cracking occurred; this is earlier than it should and was also at a lower deflection than normal (but at normal peak-load level). From the point of cracking onward, the deformation rate stabilized at the correct speed at 1 mm/min. The extent to which these early deviations in the test procedure have influenced the results is not clear, but it is notable that the square panels that suffered such a procedure performed poorly with regard to EAC.

5.3 Energy absorption capacity

In principle, we believe the round and square methods are close to equal, hence in theory we expect the correlation factor for EAC to be 1. This would indeed be the case for the situation shown in Figure 6(a) for four cracks being perpendicular to the support. But square panels can fail by a crack pattern that approaches the case in Figure 6(b), which results in 40% less total crack rotation. The sensitivity to the crack orientation implies a higher coefficient of variation (COV) for the square panels, something which is also the case in these tests (on average). The inherent scatter in results for each panel method can actually explain the variable correlation factor, from 0.7 to 1.1, which was found in the different sets of parallel sets of round and square panels. Note that Figure 6 does not show all possible crack situations that are apparent in Figure 4.

Assuming that the two panel methods are equal, hence among a large number of parallel sets the average correlation factor is 1.0. The overall average COV for EAC was here found to be 10% (0.1),

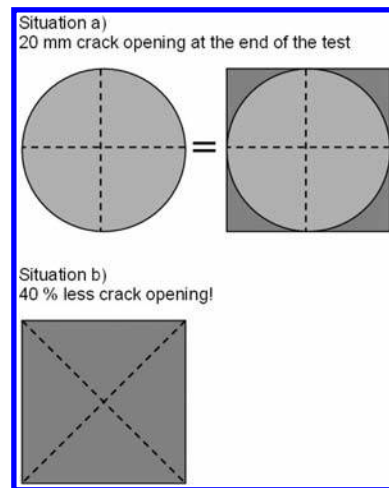


Figure 6. Possible crack patterns, four perpendicular cracks.

thus the expected “standard” interval for the correlation factor between square and round panels will then be from 0.82 to 1.22, see Equations 1 and 2.

$$\frac{EAC_{\text{Round}}}{EAC_{\text{Square}}} = \frac{1 - COV}{1 + COV} = \frac{1 - 0.1}{1 + 0.1} = \frac{0.9}{1.1} = 0.82 \quad (1)$$

$$\frac{EAC_{\text{Round}}}{EAC_{\text{Square}}} = \frac{1 + COV}{1 - COV} = \frac{1 + 0.1}{1 - 0.1} = \frac{1.1}{0.9} = 1.22 \quad (2)$$

All correlation factors from all parallel sets of round and square panels are shown in Figure 7; from the left to the right:

- Series 1, NPRA, 20 kg steel fibres
- Series 1, SINTEF, 20 kg steel fibres
- Series 2, 20 kg steel fibres
- Series 2, 35 kg steel fibres
- Series 3A, $w/(c + 2s) = 0.40$, 15 kg steel fibres
- Series 3A, $w/(c + 2s) = 0.40$, 30 kg steel fibres
- Series 3A, $w/(c + 2s) = 0.40$, 45 kg steel fibres
- Series 3B, $w/(c + 2s) = 0.40$, 3 kg macro pp-fibres
- Series 3B, $w/(c + 2s) = 0.40$, 5.5 kg macro pp-fibres
- Series 3B, $w/(c + 2s) = 0.40$, 8 kg macro pp-fibres
- Series 3A, $w/(c + 2s) = 0.39$, 30 kg steel fibres
- Series 3B, $w/(c + 2s) = 0.39$, 5.5 kg macro pp-fibres

The black dashed line at 1.0 indicates “expected behaviour”. The “standard interval” is also indicated by grey dashed lines at 0.82 and at 1.22. It can be seen that most of the correlation factors lay within the “standard interval”, but some fell outside; which also can be expected from a statistical viewpoint.

As stated earlier, the average correlation factor for all sets was 0.91. This means that the square panels gave on average 9% lower EAC than the round ones. As discussed earlier the loading rate was somewhat different for some of the square panels in Series 3 A. If these panels were removed from the data two of the square sets would be completely eliminated (“ $w/(c + 2s) = 0.40$ and 45 kg steel fibre” and “ $w/(c + 2s) = 0.39$ and 30 kg steel fibre”) as well as one panel in the set “ $w/(c + 2s) = 0.40$ and 30 kg

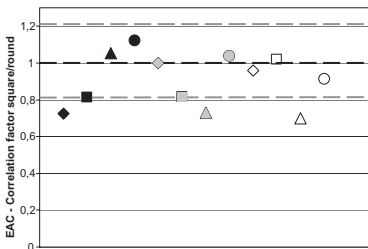


Figure 7. Energy absorption capacity, correlation factor square/round, all series.

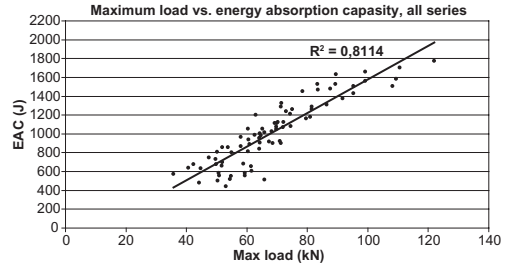


Figure 8. Maximum load versus energy absorption capacity, all series.

steel fibre”. When eliminating the deviating panel test from the latter square panel set the correlation factor increases from 0.82 (see Table 5) to 0.88. With this correlation factor for this particular set, as well as the complete elimination of the other two square panel sets, the average correlation factor for the remaining 10 parallel sets becomes 0.95.

Another peculiarity is that in sets of nominally identical panels there is a trend that a high EAC from a given panel test is associated with a high peak-load, see Figure 8. This interaction is surprising since the former should be governed mainly by the fibre action, while the latter is governed by the matrix properties. Note that this is not the case for Series 1, but applies to Series 2 and 3 individually, and all sets put together. This might indicate that fibre performance was related to concrete strength for the present mixes.

6 CONCLUSION

Spraying, trimming, storing and handling (including saw-cutting) of 1×1 m square panels is difficult and time-consuming. Trimming of such sprayed concrete panels (Series 1) was only possible with a reduced accelerator dosage. We recommend that square panels should be sprayed to their final size (as done for round panels), assuming that the influence of edge-effects (rebound) is minor. Steel moulds should be used for both square and round panels in order to cope with the weight of the concrete and to be stable over time with regard to handling and moisture. A minimum of 4 mm steel sheet, as required in EN 14488, should be used for both square and round panels. This ensures that the degree of flatness of the panels is as high as possible. Any panel unevenness will lead to point contact with the support fixture and thus the support will not be continuous, something which will influence the crack propagation and crack pattern. For the same reason we recommend that panels should be ori-

ented with the smooth moulded face against the support fixture (the orientation of round panels have traditionally been the opposite).

In the present investigation a steel frame bedded on thin strips of a hard and smooth type of chip-board was used as support fixtures for the square panels, whereas a wooden ring (no bedding material) was used as the support fixture for the round panels. The use of a bedding material of mortar or plaster, as described for square panels, is considered too impractical for routine testing.

Energy absorption capacity (EAC) test results should be corrected for panel thickness. For the present results this has been done using an analytical approach. Naturally, such corrections can also be performed using an empirical approach. The correction procedure assumes that at a given crack opening the moment intensity in a crack is linearly dependent on the panel thickness. The procedure therefore corrects for thickness and modifies the final displacement level at the end of the test in order to ensure that the crack opening interval from start till end is the same in all tests. The procedure assumes 4 cracks, and thus do not take into account that the number of cracks can sometimes be more. We have no data that documents the area of application, but assumes that it is valid for relative small variations in panel thickness; such as the variations reported in this paper.

It is expected that the square and round panel methods should give about the same EAC-result; hence in theory, the correlation factor (EAC from square panels divided by EAC from round panels) is believed to be 1. But in practice the result from the square panel may be affected by the crack orientation. The results from the sets of parallel testing of square and round panels with nominally identical fibre reinforced concrete show that the correlation factors varied from 0.70 to 1.12 and that the average correlation factor for all parallel sets (12 in total) was 0.91. The variability of the correlation factors must be expected with the given scatter of the two panel methods. The average coefficient of variation among all sets with square panels was 12.4% and for the round panels the variation was 7.6%.

Within a set of panels there is a trend evident that a higher EAC from a given panel test is associated with a high peak load, and vice versa. This is surprising since the former should be governed by fibre actions while the latter is a property of the concrete matrix, and both fibre dosage and concrete mix is the same within each set.

Visual observations during the tests showed that the panels transfer friction forces to the support fixture. Friction forces to the base will influence the measured EAC. The friction effect has been quantified in later tests.

ACKNOWLEDGEMENT

The investigation was undertaken as a part of the on-going standardization work of the Sprayed Concrete Committee of the Norwegian Concrete Association. The discussions in the committee are acknowledged. So is the contribution from the contractor Entreprenørservice AS for planning and production of specimens, as well as transport. All testing in the NPRA-laboratory has been done in close cooperation with the permanent staff at the laboratory. This cooperation is also greatly appreciated.

REFERENCES

- Bjøntegaard Ø. & Myren S.A. 2008. Energy absorption capacity for fibre reinforced sprayed concrete. Effect of panel geometry and fibre content (Series 2). Technology report no. 2532, Norwegian Public Roads Administration (in Norwegian).
- Bjøntegaard Ø. 2009. Energy absorption capacity for fibre reinforced sprayed concrete. Effect of friction in rounds and square panel tests with continuous support (Series 4). Technology report no. 2534, Norwegian Public Roads Administration.
- EN 12350-2 Testing fresh concrete—Part 2: Slump test, 2000.
- EN 12350-7 Testing fresh concrete—Part 7: Air content—Pressure methods, 2000.
- EN 12390-3 Testing hardened concrete—Part 3: Compressive strength of test specimens, 2001.
- EN 12390-7 Testing hardened concrete—Part 7: Density of hardened concrete, 2001.
- EN 14488-1 Testing sprayed concrete—Part 1. Sampling fresh and hardened concrete, 2005.
- EN 14488-5 Testing sprayed concrete—Part 5. Determination of energy absorption capacity of fibre reinforced slab specimens. 2006.
- EN 14488-7 Testing sprayed concrete—Part 7: Fibre content of fibre reinforced concrete, 2006.
- Handbook 014 Laboratory investigations. Guidelines. The Norwegian Public Roads Administration, 1997 (in Norwegian).
- Myren S.A. & Bjøntegaard Ø. 2008. Energy absorption capacity for fibre reinforced sprayed concrete. New test rig, effect of panel geometry and testing laboratory (Series 1). Technology report no. 2531, Norwegian Public Roads Administration (in Norwegian).
- Myren S.A. & Bjøntegaard Ø. 2009. Energy absorption capacity for fibre reinforced sprayed concrete. Effect of panel geometry, fibre content and fibre type (Series 3). Technology report—to be published in 2009.
- Norwegian Concrete Association's publication no. 7 (NB 7): Sprayed concrete for rock support. 2003 (in Norwegian).
- Thorenfeldt E. 2006. Fibre reinforced concrete panels. Energy absorption capacity for standard samples. SINTEF memo (in Norwegian).

Shotcrete application on the Boggo Road Busway driven tunnel

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ABSTRACT: The Boggo Road Busway Project includes 430 m of driven tunnel. The tunnel, with an excavated width of 15 m, was constructed beneath the heritage listed Boggo Road Jail with low ground cover in very variable geological conditions. Shotcrete was the main initial arch support type relied upon to minimize surface ground settlement through this shallow section of tunnel. To achieve a high arch stiffness the shotcrete was applied to its full thickness of 350 mm as near as practical to the tunnel face. Other support types used along this section of tunnel were an array of canopy tubes, steel lattice girders and fibre glass face nails. Beyond the jail initial tunnel arch support types consisted of shotcrete and rock bolts or shotcrete and lattice girders. Fibre glass face nails were used along the full tunnel length for face stability.

1 INTRODUCTION

The driven tunnel examined in this paper forms part of the Boggo Road Busway project in the Queensland capital, Brisbane. This paper describes a number of aspects of the design and construction of the driven tunnel with an emphasis on the application of shotcrete for tunnel support. The driven tunnel is 430 m long with an excavated width of 15 m and a full tunnel excavation height of 8 m. The first 120 m long section of the driven tunnel was excavated under the heritage listed Boggo Road Jail. The main jail buildings and perimeter walls were built in 1908 and are of brick construction.

Ground cover over the tunnel beneath the jail site varied from 5.5 m to 8 m and as a consequence the predicted settlement values and their potential to cause damage to the buildings were critical to determining and obtaining approval for the selected tunnel alignment. The geology along the driven tunnel alignment was extremely variable, providing mixed face tunneling conditions and ranging from surface residual soils to high strength rock at depth. The maximum ground cover above the driven tunnel is 20 m. The initial ground support included shotcrete, and shotcrete was also relied upon as the permanent support at the four 19 m long jet fan niches and at both driven tunnel portals (in combination with permanent rock bolts). The remainder of the tunnel permanent support consists of steel reinforced 300 mm thick concrete over the tunnel arch (the tunnel has a horseshoe shaped profile, [Figure 1](#)). Where shotcrete was used as permanent support the water-proofing membrane was a spray on product,

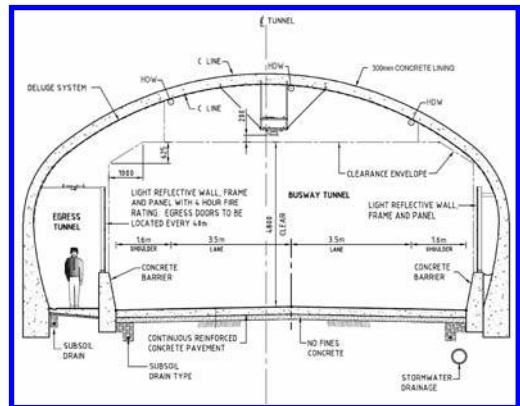


Figure 1. Standard tunnel section through completed tunnel.

in contrast to the sheet product used where there was an in-situ concrete lining.

2 DESCRIPTION OF THE PROJECT

The Queensland Government funded A\$326 million Boggo Road Busway is a dedicated public transport link to the South East and future Eastern Busway at Buranda and provides the connection from the Eastern and Southern Busway corridors to the University of Queensland with a total travel length of 2 km.

The Boggo Road Busway commences at the new Princess Alexandra (PA) Hospital busway station and continues through a cut and cover underpass



Figure 2. Plan of Boggo Road Busway and Eastern Busway alignment, driven tunnel on left below bus turning bay.

beneath the Queensland Rail lines, then travels parallel to the railway lines to reach the new Park Road busway station and existing Park Road rail station (Figure 2). From this point the busway goes underground through a 630 m long tunnel that passes beneath the Boggo Road Jail, Gair Park and Annerley Road before emerging at the new 330 m long cable-stayed Eleanor Schonell Bridge in Dutton Park which then connects across the river to the University of Queensland. At its eastern end the Boggo Road Busway connects to the South East Busway via the first section of the Eastern Busway which starts at the PA Hospital and travels over Ipswich Road and under the Pacific Motorway.

The Boggo Road Busway was opened in July 2009 and will be used by 600 buses daily (this equates to around 13,000 passengers). The Boggo Road Busway project and the first stage of the South Eastern Busway have been designed and constructed using the Alliance method of project delivery. The Alliance team members are Queensland Transport (the client), Thiess Contractors (the builder) and Sinclair Knight Merz (the designer). The route alignment is shown in Figure 2 above with the tunnel alignment on the left side of the figure (Nye et al, 2009).

3 DESCRIPTION OF THE DRIVEN TUNNEL

The busway tunnel's full length is 630 m and consists of 430 m of driven tunnel with cut and cover tunnels at both ends of the driven tunnel. At the north end, the cut and cover tunnel is 130 m long and includes a 34 m diameter bus turnaround area

at the driven tunnel portal. At the south end of the driven tunnel the cut and cover section is 70 m long and daylights into Dutton Park.

The busway tunnel profile includes 2×3.5 m bus lanes with 1.6 m shoulders on both sides of the busway together with a 1.5 m wide emergency egress passage along one side of the tunnel. The road pavement to tunnel crown is around 7 m in height. Four jet fans niches are spaced along the tunnel, each required over-excavation of the tunnel crown to accommodate the three jet fans at each fan niche location. The jet fan niches were completed outside the jail buildings but within the driven tunnel. The final lining of the driven tunnel consists of a 300 mm thick in-situ reinforced concrete arch lining for the main running tunnel. At the fan niches, the final lining consists of a pattern of permanent rock bolts and 250 mm of steel fibre reinforced shotcrete.

The waterproofing membrane behind the in-situ concrete lining consists of a two color layer 2 mm thick polyethylene welded sheet over the arch and upper wall of the standard tunnel profile. At the fan niches and tunnel portals a 4 mm thick spray on membrane has been used. The driven tunnel has been designed to be fully drained and has a "no fines" concrete drainage layer under the reinforced concrete road pavement as part of the under pavement drainage system. The tunnel was excavated using an AM105 road-header.

4 GEOLOGICAL MODEL

The geological profile for both the rock type and rock strength varies considerably along the length

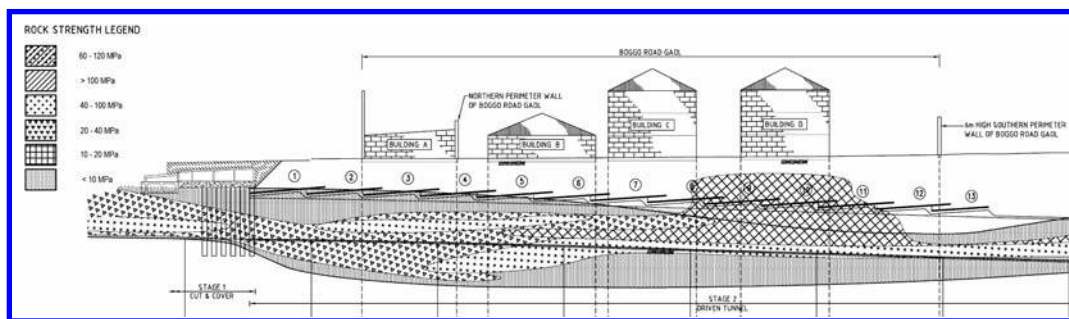


Figure 3. Geological profile under the jail.

of the driven tunnel and across the tunnel section. Developing the geological model profile was quite difficult because of the varying geology. To develop the model required a good understanding of the local geological history as well as the ability to extrapolate the data in three dimensions. It was realized in the design development stage that the geological model would have a significant impact on the selection of tunnel support types along the tunnel and hence the actual cost of the tunnel.

There are industry recognized classification systems which also provide recommended ground support for a given rock mass classification. These include the Q and the Rock Mass Rating (RMR) systems. An initial assessment using both these systems was carried out with the RMR system classifying the ground along the tunnel alignment in the range of 'poor' to 'good' rock conditions. The purpose of the geological model was to provide a prediction of the variation of the rock strata along, across and above the tunnel (Figure 3). Rock strength data was superimposed on the geological model. It was this latter information that was used to develop a very simple site-specific rock support classification system and consequently this was used to determine the extent of each support type along the tunnel. For example, the fully encapsulated resin rock bolts could not be used as initial ground support if the UCS of the rock was less than 10 MPa as this would compromise the desired bond strength along the rock bolt. Elastic Modulus values for rock used in FE analyses for lining design and settlement predictions were in the range of 200 MPa to 5000 MPa depending on the rock type and degree of weathering.

5 INITIAL TUNNEL GROUND SUPPORT

The design of the initial ground support for the tunnel is briefly described below. During construction very few modifications were made to the initial design of the initial tunnel support (Table 1). One

Table 1. Initial tunnel support types.

Type	Description of tunnel initial ground support
1	3.6 m long 21 mm diameter, M24 thread, 28 mm hole, 310 kN ultimate capacity fully resin encapsulated rock bolts on a 1.5 m grid, 50 mm thickness shotcrete. To be used in "good rock" where >80% of rock over the arch is high.
2	As above but 1.2 m grid and with 100 mm shotcrete (also modified during construction to 1.2 m (over) by 1.5 m along tunnel grid). To be used in "fair rock" where >50% of rock over arch is high strength.
3	As above but 150 mm shotcrete. To be used in "poor rock" where >80% medium strength rock over tunnel arch and UCS >10 MPa.
4	Triangular profile lattice girder at 1.2 m spacing (170 mm deep section) with 300 mm shotcrete. To be used in "very poor rock" where UCS less than 10 MPa.
5	Alweg 140 mm diameter canopy tubes at 500 mm spacing (27 no.) over tunnel arch. The canopy tubes are 12 m long with 3 m over lap, plus triangular lattice girder 170 mm deep (with 350 mm shotcrete). Low cover tunneling under Boggo Road Jail. The tunnel width was kept constant, i.e. the canopy tubes were only angled up vertically not horizontally.

significant change during construction was the deletion of steel fibres from the shotcrete for Support Types 1 to 3. This was possible because the ground behavior over the tunnel arch was better than expected. In contrast the face stability was of more concern, as evidence by an increase the number of fibre glass dowels installed at the tunnel face. One localized face collapse also occurred during tunneling, justifying the inclusion of the face nails. Variations in the length of the support types over the tunnel arch from the original design estimates occurred along the tunnel outside the jail section (Table 2). Rock strength data principally consisted of Point Load Test (PLT) and laboratory Unconfined Compression Tests (UCS) results.

Table 2. Initial tunnel ground support type—predicted and actual.

Type	Percentage of tunnel length	
	Predicted	Actual
1	20	5
2	16	14
2 modified	0	33
3	5	0
4	32	21
5	27	27

On this project a multiplication factor of between 17 (welded tuff) and 20 was used to estimate the UCS of the rock from the much larger test sample of PLT results.

In the design phase of this project careful assessment of the ground support requirements were conducted over very short lengths of the tunnel due to the wide range of geotechnical parameters. The geological model and the superimposed strength data were continuously reviewed throughout the design and subsequent construction phase. This review resulted in two additional boreholes in Annerley Road (after tunnel construction had commenced), daily geological mapping data from the tunnel, additional field PLTs on rock samples taken from the tunnel, and regular pullout tests on rock bolts in the tunnel walls and crown as tunnel excavation progressed.

The reason for the “rigid” initial Support Type 5 was to minimize ground relaxation due to excavation of the tunnel and hence prevent damage to the heritage listed brick building of the Boggo Road Jail. The predicted settlement was 10 mm and the actual settlement under the buildings ranged between 7 mm and 12 mm with no damage to any of the building or the perimeter 6 m high jail walls. The steel lattice girder provided a visual tunnel profile upon which a check could be made regarding the accuracy of the excavation to achieve the design profile and also provide roof early protection until the shotcrete was applied. The shotcrete was the main structural feature limiting the settlement of the surface once the 3D effect of the canopy tubes at the tunnel face was completely lost, probably after the tunnel had advanced only a few metres. This was the reason why it was necessary to apply the shotcrete at the tunnel face at near its maximum required thickness of 350 mm. Extensive finite element modeling was carried out during design development to determine the required minimum shotcrete thickness. The estimate of support for each type and the actual percentage used is given in Table 2 above.

Type 5 support under the jail was not modified. Type 4 had the shotcrete thickness reduced

to 200 mm thickness after monitored settlement readings were less than expected beyond the jail. Steel fibres at 45 kg/m³ were deleted from supports Types 1 and 2 saving A\$3/kg which became a very significant saving for the project. Type 2 modified support was used for the majority of the length of tunnel beyond the jail.

The fan niches had the same arch profile as the standard tunnel profile, and therefore had the same arch support for given geological conditions. The fan niche length of 19 m also includes the 10% graded transitions from the standard tunnel profile, required for the efficient operation of the jet fans.

6 SHOTCRETE STRENGTH AND TESTING

Under the jail the design intent was to provide initial tunnel ground support that would not allow ground relaxation. This was achieved using a staged construction approach, including forward installation of canopy tubes, steel lattice girders, 12 m long fibre glass face dowels and then the early application of the 350 mm thick shotcrete over the arch to be built up progressively at the tunnel excavation advanced in 1.0 m increments (refer Figures 9, 10 and 11).

Early strength of the shotcrete was critical to the success of this approach as the shotcrete was the stiffest structural element once the 3D constraining effect of the tunnel rock face and near face support (canopy tubes, lattice girders and face nails) was lost. Table 3 above is from the project shotcrete specification and gives the strength requirements of the shotcrete for example from as early as 3 hours after application, together with the

Table 3. Type 5 support, early and minimum strengths specified, plus calculated E-shotcrete.

Age	Minimum strength (MPa)	Test applied	Elastic modulus (MPa)
3 hours	1 MPa	Initial Strength Meyco Needle Penetrometer	1,000
12 hours	6 MPa	Sprayed Beam Compression (ASTM C116)	12,000
24 hours	18 MPa	Core Compressive Strength (AS1012)	20,000
3 days	26 MPa	Core Compressive Strength (AS1012)	24,000
7 days	35 MPa	Core Compressive Strength (AS1012)	28,000
28 days	40 MPa	Core Compressive Strength (AS1012)	30,000

test used to measure the shotcrete strength. The stiffness of the shotcrete, in the form of the Elastic Modulus, has been calculated using the following AS3100 formula at Clause 6.1.2.

$$E_c = 0.043 \rho^{1.5} \sqrt{f'_c} \quad (1)$$

in which ρ is the density of the concrete in kg/m³ and f'_c is the compressive strength. Referring to Table 4, the initial strength gain of the shotcrete in the first 12 hours was consistently good, generally meeting the required specification target strengths and hence stiffness. Although there were some quality issues with the shotcrete supply with core strength sometime very low there was no apparent negative impact on the surface settlement (further comparisons between early strength testing of shotcrete can be found in Clements, 2004).

With settlements in the range of 7 mm to 12 mm under the two jail cell block buildings the dominating variable that was the causing this difference was the varying geological profile. These two identical buildings, C and D shown on Figure 3, each are equal to a dead load over the tunnel of 60 kPa or 3 m height of fill. The surface settlement prediction in open ground was between 6 mm and 10 mm. A lot of effort went into investigating the variability of the shotcrete strength during construction but at no stage was the construction progress impacted upon. The investigations and testing consisted of changing the shotcrete mix design, varying the percentage of accelerator and obtaining test data from an alternative supplier. Apart from the tunnel arch, shotcrete also provided a continuous footing at the springline level of the tunnel heading. Where highly weathered claystone was encountered a localized widening of the base of the shotcrete arch was carried along one side of the tunnel. This is further described in Section 7.

Table 4. Minimum and maximum strength range of shotcrete for various age ranges and tests.

Age	Days	Spec (MPa)	Core strength*		Cyl. strength*	
			Min	Max	Min	Max
3 hrs	0.125	1	0.6 (MP)	1.2 (MP)	0.6 (MP)	1.2 (MP)
12 hrs	0.5	6	5.2 (B)	7.3 (B)	5.2 (B)	7.3 (B)
24 hrs	1	18	7.33	31.33	16.5	32
3 days	3	26	12	35	34	48
7 days	7	35	17	48	43	55
15 days	15		18	49	49	65
28 days	28	40	27	65	51	94

* Unless otherwise noted, MP = Meyco Needle Penetrometer, B = Sprayed Beam Compression.

There were three shotcrete specifications issued for the tunnel project. One specification was issued for each of the temporary and initial shotcrete, with and without steel fibres. The third specification was for the permanent shotcrete with steel fibres. The permanent shotcrete was used at the fan niches and at both tunnel portals. The Department of Main Roads, Queensland required that all permanent shotcrete have a minimum of 20% fly ash. The specified minimum characteristic strength of all shotcrete was 40 MPa, with maximum of 10 mm aggregate size and slump of 120 mm. The initial shotcrete layer did not require a smoothing layer prior to the installation of the waterproofing membrane sheet. This was partially due to the skill of the applicators of the shotcrete and also to the adoption of a 10 mm maximum aggregate size. The polyethylene sheet was also specified to be fitted with a geotextile backing layer for additional protection against penetration damage.

One of the options considered for the final lining during design development was to use a combined lattice girder and shotcrete lining and not to have an in-situ concrete lining. There were potential cost savings and productivity gains with this option compared to using an in-situ concrete lining. Shotcrete shadow trials (Figure 4) on test panels with sections of lattice girders did not prove satisfactory because it could not be demonstrated that the reinforcement would be fully embedded in shotcrete (Figure 5). This is potentially a long term durability issue. In the final design the fan niches consisted of steel fibre reinforced shotcrete used in combination with a pattern of permanent rock bolts. The pattern of rock bolts also included the 21 stainless steel rock bolts used to support the three jet fans located in each fan niche.



Figure 4. Lattice girder shotcrete test panel.

Both the north and south portals were also supported by a combination of steel reinforced shotcrete and permanent rock bolts or dowels. At both the four fan niches and at the two tunnel portals the reason for using shotcrete was for construction

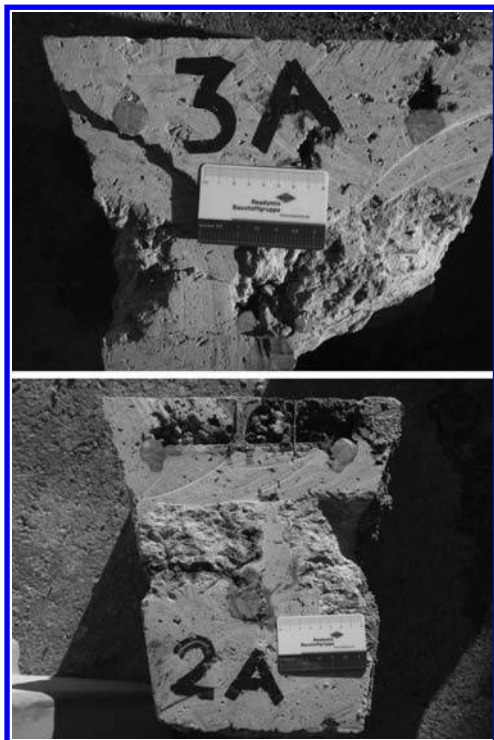


Figure 5. Test panel sections showing voids.

efficiency which also saved costs compared to the alternative of providing formwork which had no or little potential for reuse. The fan niches required an over-excavation of the tunnel crown to accommodate the three 1200 mm diameter jet fans and did not therefore suit the steel formwork of the standard tunnel profile.

Figure 7 shows the application of a spray-on waterproofing membrane at a fan niche location. The spray-on membrane was adopted only after a number of field trials were carried out. Initial trials were also carried using the polyethylene sheet water-proofing membrane (also shown in Figure 7), and proved unsuccessful because the shotcrete would not bond to the polyethylene sheet, for two reasons. Firstly, there was no adhesion, and secondly the sheeting had air voids underneath, creating an unstable surface area on which to shotcrete.



Figure 7. Fan niche waterproofing application.



Figure 6. Shotcreted tunnel portals, north portal shown.

The spray-on membrane required a minimum drying time of 24 hours between layers, much longer than the supplier's estimate of 6 hours.

All shotcrete was applied using a Meyco Suprema robotic rig. Shotcrete and concrete was delivered to the tunnel via two drop pipes located near the north end of the tunnel. Two off-road concrete agitators for use in the tunnel were purchased for the project. Concrete and shotcrete could then be transported efficiently in all weathers and without subcontractors being required to enter the tunnel (Bradford, 2008).

Shotcreting conditions in the tunnel were good with very little or no groundwater ingress impacting on the shotcrete during application. If there was any ground water present this was channeled away to the tunnel invert leaving a dry surface for shotcreting. The tunnel portals initially were established with temporary rock bolts, permanent dowels and an initial shotcrete layer. The permanent support consisted of a pattern of permanent rock bolts, followed by the application of the spray-on membrane, followed by an initial 25 mm of shotcrete without steel fibres, followed by the permanent steel fibre reinforced shotcrete. From Figure 6 above it can also be readily seen that it was possible to include a drainage channel (with a half PVC pipe as the form) across the portal and a vertical niche for the latter matching of the vertical emergency egress side panel from the adjoining cut and cover tunnel (refer to Figure 1). The design life specified for all permanent structural support was 100 years.

7 CONSTRUCTION UNDER THE JAIL

Previous sections of the paper have referred to shotcrete and the other support used under the jail. This section provides more details of the construction sequence aided by a number of sketches which together show the application of shotcrete as used in the shallow cover tunnel section. The aerial view in Figure 8 shows the tunnel alignment under the jail.

Developing the construction sequence under the jail required a lot of forethought and analysis. Two previous examples of shallow tunnel excavations in Brisbane had taken different approaches. In particular, the Vulture Street Busway tunnel completed in 2000 used pre-excavated side drift tunnel filled with mass concrete to create the arch footings. The Buranda Busway tunnel competed at the same time recorded settlement values under over-lying railway tracks of around 20 mm and with less ground cover. Smaller but pre-installed footings were used along the arched section. In the end there was a clear difference in the Buranda design compared



Figure 8. Aerial view of tunnel alignment under the jail.

to that adopted at Boggo. The principle difference was that at Boggo we adopted lighter lattice girders but much thicker shotcrete (350 mm compared to 200 mm thickness over the tunnel arch) and a wall beam, as an arch footing, was only installed as required. To limit the settlement to the predicted values the FE analyses highlighted the sensitivity of the shotcrete stiffness properties on these predictions. This is why during shotcrete trials and construction so much effort was placed on obtaining the early strength of the shotcrete and why a very prescriptive construction sequence was provided on the design drawings for the construction team to follow. In practice the construction team was able to achieve more than specified in Table 5 in terms of applying the full 350 mm of shotcrete near to the tunnel face.

The rate of construction is also an important parameter and the actual excavation rate under the jail was around 9 m per week. This excavation rate was more than adequate to allow the shotcrete to gain sufficient early strength and hence sufficient stiffness. The canopy tubes had the capacity to span alone for 3 m back from the face without shotcrete support (Figures 9, 10 and 11). This was equivalent to at least 2 days excavation production. Referring to Table 3, it can be seen that at 2 days the shotcrete would have attained a stiffness at least above $E = 20,000$ MPa (which is well above the average stiffness of the surrounding rock). In any parametric studies, shrinkage and creep values of the shotcrete are almost insignificant given our real knowledge of the actual and obvious wide range of rock mass properties including its stiffness. Behind the tunnel face, after the shotcrete had reached near full strength, the actual stresses in the

Table 5. Tunnel heading excavation and support sequence Type 5 (tunneling under jail).

Sequence No.	Description of excavation and installation support
1	The tunnel is to be excavated by heading and bench.
2	For the heading excavation install an array of 12 m long cement grouted steel canopy tubes from the tunnel over the tunnel arch.
3	In very low to low strength rock install a 1 m grid of 12 m long cement grouted fibreglass dowels.
4	Advance the tunnel heading in 1 m increments. The stability of the face will be continuously assessed by tunnel/geotechnical engineer with each excavation cycle.
5	Stand the lattice girder and holding in position for shotcreting using steel or similar brackets attached to the lining behind the face or welded to any exposed canopy tube, and or by timber blocking.
6	The excavation cannot advance until the lattice girder has been fully embedded in shotcrete as described in Items 7 and 8 below.
7	Up to and including the lattice girder expansion joint in each side of the tunnel fully embed the lattice girder footing and lattice girder arch in shotcrete. A minimum thickness of 200 mm of shotcrete is to be applied over this section of the arch profile between lattice girders.
8	Over the remaining crown arch profile apply 80 mm of shotcrete over any exposed ground between lattice girders and fully embed the last erected lattice girder in shotcrete with 25 mm minimum cover on the inside reinforcement bar(chord B1).
9	As the tunnel advances apply up to 90 mm layer between lattice girders until the minimum uniform 350 mm thickness of shotcrete is achieved behind the tunnel face. i.e. the thickness of shotcrete within 3 m of the face shall not be less than 350 mm before commencing the next excavation cycle.
10	Ensure the shotcrete surfaces including and circumferential joints formed are clean of any debris before applying shotcrete in the next excavation cycle.
11	When the tunnel heading excavation has advanced 9 m repeat items 2 to 10.

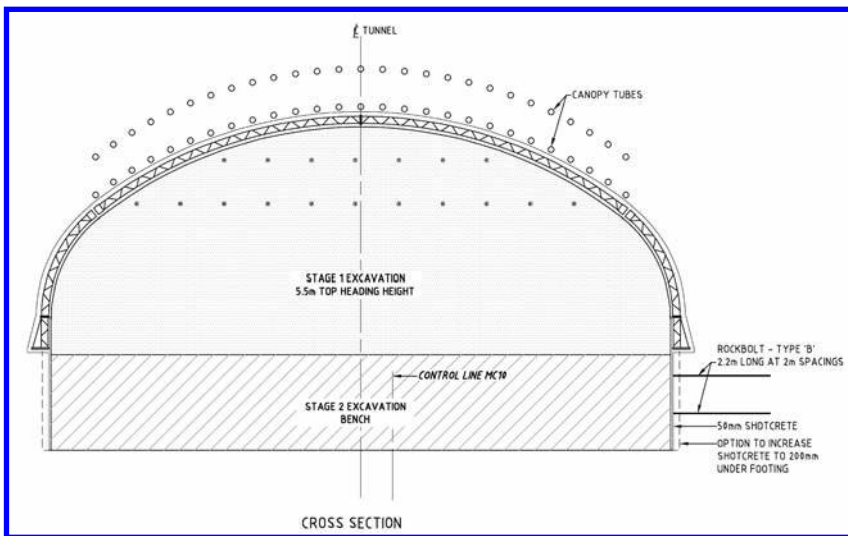


Figure 9. Cross section through the tunnel showing 27 canopy tubes over the tunnel arch.

shotcrete were very low, even with a full overburden loading of around 5 m to 8 m of ground cover, thereby negating creep effects. The shotcrete arch under the jail was also reinforced with steel lattice girders, spaced at 1 m centres. Even considering the rock property uncertainties, the steel lattices girders alone would have negated any potential shrinkage effects of the shotcrete on surface settlement, even if this phenomena were significant.

The widened footing details shown in Figures 12 and 13 were used along the LHS side of the tunnel for a length of 40 m starting from Cell Block Building C (Figure 2), when the tunnel jail intersected highly weathered claystone at the level of the arch springline. The UCS of the claystone rock would have been less than 5 MPa. The sequence of reinforcement and shotcrete installation shown in Figure 13 worked extremely well. The intent of the

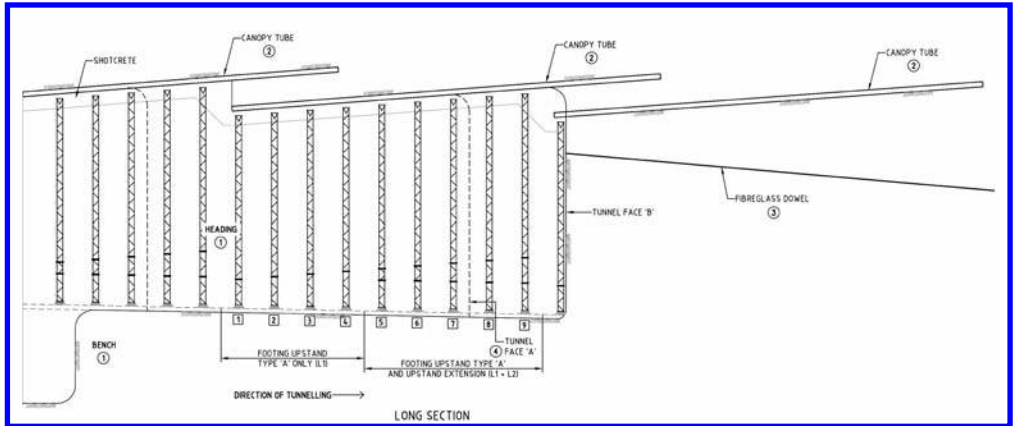


Figure 10. Long section of the tunnel with lattice girders and canopy tubes shown.

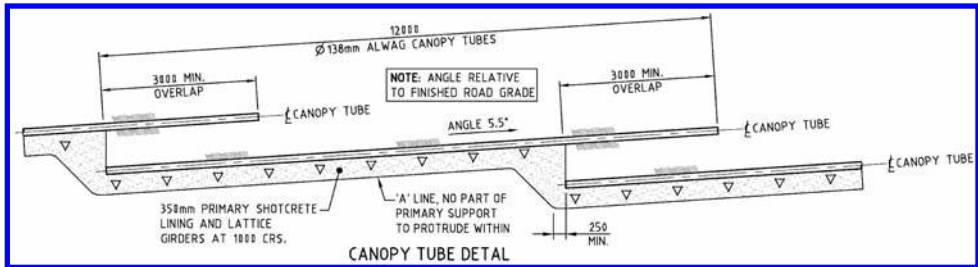


Figure 11. Details of the canopy tube and shotcrete in a section through the tunnel crown.

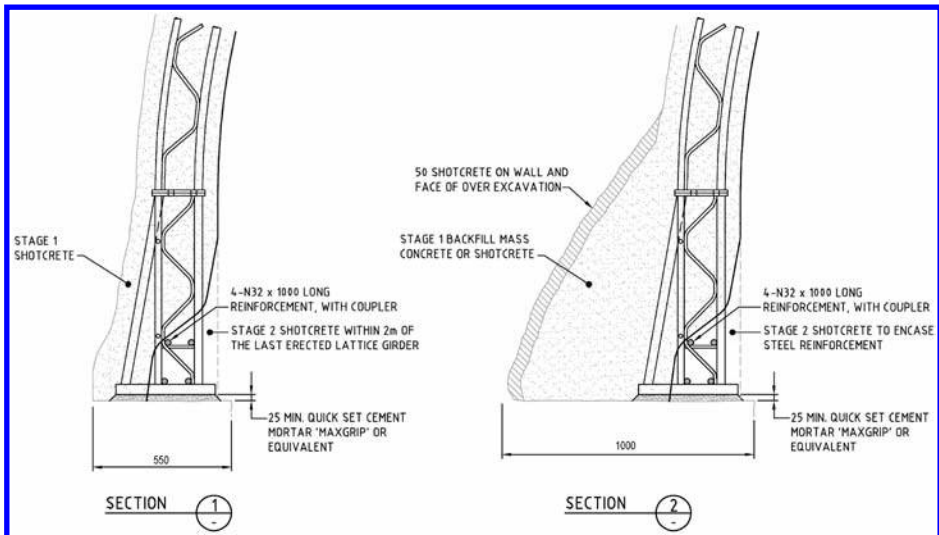


Figure 12. The standard shotcrete and alternative wider footing/wall beam for highly weathered claystone.

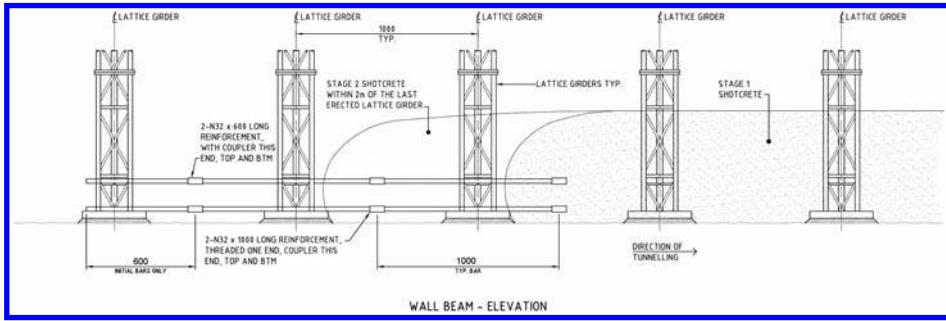


Figure 13. Longitudinal section along wall beam showing the shotcreting sequence.

steel reinforcement was to create a temporary beam prior to subsequent bench excavation to allow the installation of an additional 200 mm shotcrete beneath the beam to act as a continuous deep footing transferring the arch load down to the invert of the tunnel. This last sequence, which is not shown here diagrammatically, was not required.

Together with the tunnel monitoring results during construction the other important tool was the 'Permit to Excavate' process. On a daily basis a team consisting of the geotechnical engineer, the designer and a representative of the construction team would meet to discuss the previous day's construction and prepare for signing a permit for the next 24 hours of tunneling. Items on the agenda included safety, face mapping, monitoring data and the reporting or otherwise of any damage to the surface buildings. Pre-construction surveys of the buildings had been carried out and various instruments were fitted to the buildings to monitor existing cracks. Visual observations were also regularly carried out, often daily, depending on the criticality of the tunneling work at the time.

Shotcrete was rarely used to stabilize the tunnel face. After face mapping the geotechnical engineer would provide a pattern for the next array of 12 m long fibre glass dowels (the dowels had the same 3 m overlap as the canopy tubes). The number of face dowels numbered over 30 on a regular basis, and were seen as a more reliable method of providing face stability than shotcrete. The exposed geology of the tunnel face varied both across the face and along the tunnel length. One face collapse did occur when it appeared that the dowels did not bond sufficiently to the surrounding rock. In some cases the dowel failed by shear and in others the rock just broke away leaving the undamaged dowel end suspended in the air. This incident resulted in a change in the installation of the lattice girders in that they were stood using four split sets installed across the tunnel arch using the robotic drilling. The footings of the lattice girders were then shotcreted so that

there was no need for anyone to place blocking at the lattice girder footing by hand. The shotcrete would set while the lattice girders were fully suspended from the tunnel arch via the split sets.

8 FURTHER CONSTRUCTION COMMENTS

The primary concerns of the construction team, in the installation of shotcrete is safety, quality, cost, productivity and effect on other processes. The construction team worked closely with the designers to achieve the desired structural outcomes with the most efficient application methods. This was achieved through initial constructability reviews held prior to commencement of a section and continuous reviews through the 'Permit to Excavate' process. Constructability reviews provided a great conduit for the designers to communicate intended sequence and outcomes and for the two teams to identify issues and innovations. The Permit process allowed for fine tuning using the added information of monitoring data.

The primary safety concerns regarding shotcrete are adhesion, fall out and how the shotcrete provides support. The high early strengths required allowed the construction team to progress to the next round without delay. The cutting, lattice girder and bolting process was achieved with a 5 m exclusion zone from the face, which allowed sufficient time for the shotcrete to reach a safe strength. Monitoring for cracks yielded very few results, which gave confidence in the support. There were no issues with adhesion due to the rough surface of the encountered rock mass. Fall out was encountered in the initial stages of attempting to spray 350 mm in one pass, however this was alleviated by mix and techniques improvements. Fall out only occurred in the area being currently sprayed, not in previous shifts areas, which also indicated sufficient bonding.

Quality targets are for a consistent product within specification and a finish that will not require a secondary treatment for the application of waterproofing membrane. Shotcrete has a number of variables which can adversely affect specification results which we found difficult to get to a final mix and application method until half way through the project. The introduction of high early strength requirements can affect other mix characteristics, as does the accelerator dosage. Intensive, prolonged testing regime had to be undertaken to come to a mix/application method that performed to expectations. The use of a 10 mm non fiber mix did meet the finish expectations for smoothness to allow for membrane installation, however this is in no small part attributed to operator experience. Other projects have shown that a 7 mm product will provide a superior finish with less skill. Lamination was present in cores to a minimal extent. This was found to be due to the accelerator injection method and was minimized by the one pass spray method.

Of course, all tunnel builders want to minimize costs. The removal of fiber from the mix reduced material costs by 1/3. All designs should be issued with a fiber/non fiber depth option as often the economics of the material supply drives towards the non fiber option. Cost improvement measures were also seen in equipment selection. The Meyco proved to be a reliable, cost effective machine with little down time and good application rates. The method of spraying up to 350 mm in one pass created productivity improvements, thus reducing costs, as did the omission of the smoothing layer due to the quality of finish. The one pass method also reduced the interaction of the spraying process with the other in tunnel activities. Smoothing layer activities, which was omitted from this project, can delay services installation.

9 CONCLUSIONS

The application of shotcrete on this project was critical to its success. Shotcrete was used to provide initial stiff tunnel support in an arch profiled tunnel to control ground settlement to within predicted and necessary limits to prevent damage to the heritage listed jail structures above.

As initial tunnel support, with 10 mm size aggregate, no special additional measures were required

to protect the sheet waterproofing membrane with its felt backing during its installation. Shotcrete together with the spray-on waterproofing membrane has also proved successful at the fan niches and at the two tunnel portals. Further research into spray-on water proofing products to reduce drying times would be useful.

Shotcrete as permanent support at the fan niches and the tunnel portals saved construction time and costs as formwork at these locations was not required. The shotcrete performed well in all of the remaining support types, 1, 2 and 4 used as initial ground support in the tunnel (Type 3 support was not used). There was also no significant, if any, cracking observed in the shotcrete along the full length of the tunnel throughout construction.

Apparently complex construction processes, such as the widened tunnel wall beam, were carried out very successfully using large localized volumes of shotcrete with no construction delays.

The shotcrete lattice girder shadow trials demonstrated that there could be durability issues with this type of construction, this is particularly so where a design life of 100 years has been specified. Obviously, there are also other important factors that could impact on the design life (e.g. ground-water chemistry).

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REFERENCES

- Bradford, G. 2008. "Brisbane's quiet achiever nears end". *Tunnels and Tunneling International*, September, pp. 23–26.
- Clements, M.J.K. 2004. "Comparison of methods for early age strength testing of shotcrete". *Shotcrete: More Engineering Developments*, Taylor & Francis, London, pp. 81–87.
- Nye, T., Kitson, M. and Chinniah, R. 2009. "Boggo Road Busway Project, Brisbane, Australia". Rapid Excavation and Tunneling Conference, Las Vegas, 14–17 June, pp. 906–915.

Use of calorimetry to select materials for shotcrete

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ABSTRACT: This paper reviews the use of isothermal and semi-adiabatic calorimetry for development of new shotcrete mixtures as well as trouble shooting existing ones. Calorimetry measures the rate of chemical reaction through the rate of heat evolution, which is easy to measure and automate unlike set and strength measurements. The measured heat or temperature curve gives the user an understanding of whether or not a given mixture design is likely to develop mechanical performance in the field or if “something has changed” that may require re-formulation for proper field performance. Modern calorimeters are easy to use, require little labor with automated data collection and if properly used can be ideal tools for assessing the effect of cement and admixture chemistry, admixture addition time, temperature as well as natural variability in shotcrete materials.

1 INTRODUCTION

Shotcrete has developed rapidly as an essential material in civil tunnel construction and as critical ground support in underground mining operations. Shotcrete used in these vital applications must perform to its full potential in order to meet its perilously important obligation of providing a safer working environment while still offering the essential performance characteristics to achieve operational efficiency. By its very nature, the vast majority of shotcrete used in underground operations is high performance concrete (HPC) with intricate mixes that are expected to exhibit intrinsically inconsistent properties, such as ~8–24 hour plastic working life, while still being able to provide very high early-age strength when ultimately placed. This type of concrete is not your everyday 20 MPa garden variety concrete. As a science, we are extending the boundaries of concrete mixture composition and introducing significant complexity.

In the underground world, it can be dangerous to assume that this complexity is not without difficulties and important responsibilities. When added together, different materials can behave in a totally unpredictable manner with dire consequences. The calorimetry methods described below have become useful tools for identifying and understanding cement-admixture incompatibility issues.

Shotcrete mix designers need to allow for the reality that materials such as cement and Supplementary Cementitious Materials (SCMs) can, by their nature, vary between sources and may also vary for the supplier and even between truck loads. Thus, the specific interactions between the complex ingredients may change. In a generic 20 MPa “slab on grade concrete”, the variations may cause some

inconvenience of delayed set times or slow rate of strength development, resulting in overtime payments or project delay costs. In the underground world, this inconvenience may lead to more significant issues. Combine the elements of varying materials with mix complexity and now add in temperature variations and the underground concrete practitioner is facing a real dilemma to explain “in-cycle” productivity issues from slower heading advancement as a result of large fallouts, slow set times and poor early age strength developments.

It needs to be pointed out that cement & SCM suppliers cannot make their products immune from these types of problems. While pre-construction testing is practiced widely in large civil tunnel infrastructure projects, it is less of a ritual in remote underground mining operations. The old adage, unless something changes, we don’t retest, is common. Ongoing testing of cement and SCMs is almost never practiced. In typical mine operations, the mine company controls the purchasing of the key shotcrete ingredients and provides a cocktail of materials to the spray operators. When issues occur, cement contents are either increased or decreased and/or admixture dose rates changed. These actions may disguise the real issues and add unnecessary cost to operations or could exacerbate the problems of material incompatibility even further.

2 CALORIMETRY

Modern calorimetry has evolved as a relatively simple and low cost method for measuring the rate of cement hydration that is responsible for set and strength development of concrete. At its simplest

form, semi-adiabatic calorimetry measures a temperature rise in an insulated sample that can be compared with results from similar samples using for example different binders or admixtures, as long as each sample has been prepared in a similar fashion. Isothermal calorimetry on the other hand measures heat flow at near-constant temperature, which allows for greater detail and accuracy, and makes testing for the effect of the all-important curing temperature easy. Note that calorimetry can only measure effects that are related to heat evolution and therefore cannot be used to characterize any phenomena unrelated to heat evolution, such as many porosity and some rheology-related issues. Whenever practical, calorimetry tests are therefore typically carried out in parallel on samples that are anyway prepared for other measurements such as set and strength development.

As calorimetry tests result in a continuous curve representing the rate of reaction of the binder, it offers additional insights into the rate of development of mechanical properties. Traditional labor-intensive performance tests are often difficult to carry out in an underground mining environment, making calorimetry a strong candidate for on-site monitoring in addition to the typical off-site use to select materials and perform mixture design optimization. Since the data recording of calorimetry tests can be highly automated, calorimetry has the potential for simultaneously monitoring of multiple samples with little additional labor (DiNoia & Sandberg 2004).

2.1 Isothermal calorimetry

Figure 1 shows a schematic design of a simple isothermal heat flow calorimeter (Wadso 2003). The heat generated from the reacting sample flows through a highly conductive heat flow sensor into a heat sink designed to absorb the heat with a minimal temperature increase. The reference is a thermally inert material with similar heat capacity as the sample. The software registers the difference in heat flow from the sample and the reference.

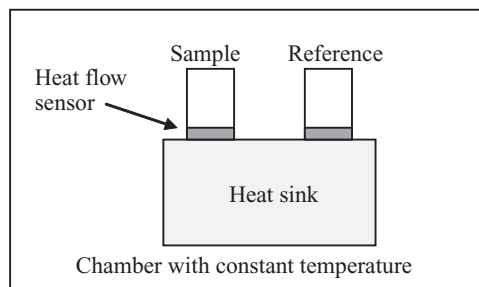


Figure 1. Schematics of an isothermal heat flow calorimeter.

Isothermal heat flow calorimetry testing of cement paste has long been used for research on the effect of additives on the hydration kinetics of portland cement (Sandberg & Roberts 2005). Traditionally, isothermal calorimetry with proper temperature control ranging from 5–60°C offers outstanding accuracy and repeatability, however the following drawbacks have limited its use in the field:

- High cost,
- Small sample size required to obtain isothermal conditions, thereby making direct studies of mortar and concrete impractical,
- Complicated interpretation of results.

More recently, multi-channel isothermal calorimeters with +100 mL samples size as well as less expensive semi-adiabatic calorimeters along with easy-to-use software applications have gained popularity because of lower cost than traditional isothermal calorimeters, easy field application, and ability to test larger samples such as concrete or mortar.

Figures 2–3 show an example of an industrial sample of an Australian General Purpose cement tested in standard ASTM C109 mortar with a fixed dose of stabilizer and water reducer, and a variable dose (as indicated) of low-alkali accelerator dosed 30 minutes after initial mixing. The detailed cement chemistry is not known and in fact not necessary since this example is designed to demonstrate the potential use of calorimetry as a screening tool for materials selection without knowledge of the detailed chemistry of the ingredients. Note the relatively linear response in Figure 2 of the accelerator as it is dosed from 2% to 8% on the broad-based cement hydration peak that relates to strength development. The accelerator is designed to induce set through rapid formation of calcium-sulfoaluminate-hydrates; this effect is

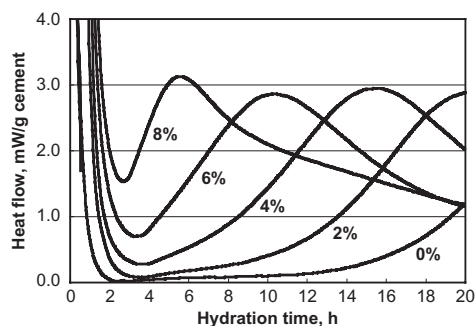


Figure 2. Example of a cement—admixture combination that responds well to an alkali free shotcrete accelerator dosed as indicated and tested for 20 h. The broad exotherm reflects primarily alite hydration. All mixtures had a fixed w/c and dose of stabilizer and water reducer according to field experience.

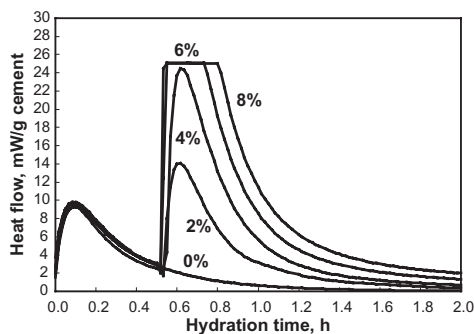


Figure 3. Effect of accelerator dosage on heat evolution immediately after accelerator dosage as indicated followed by 30 s mixing. The exotherm reflects the formation of calcium-sulfoaluminate hydrates induced by the accelerator. The plateau at 25 mW/g represents the maximum measuring capacity of the instrument.

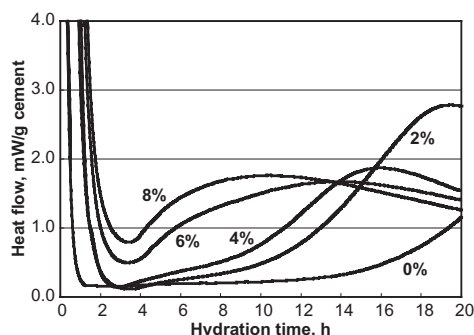


Figure 4. Example of a cement—admixture combination that responds not as well to an alkali free shotcrete accelerator as shown in Figure 2.

also visible in the calorimetry tests as shown in Figure 3.

Not all cements respond as well as the example shown in Figure 2. Figure 4 shows an example of an industrial sample of an Australian General Purpose cement that was known in the field to need a higher dose of accelerator for equal early strength in a similar mixture design as tested above (mechanical and flow data measured in the field not available for publication). Note the much lower heat evolution of this cement; compare Figure 4 with Figure 3.

2.2 Temperature effects by isothermal calorimetry

One of the most useful applications of isothermal calorimetry is to measure the effect of curing temperature on the rate of hydration of the binder, which in turn controls the strength development rate. The mixing and curing temperature may

impact both setting and the strength development rate of the shotcrete. The activation energy concept has been successfully used for normal Portland-cement based concrete and is also a potentially useful way to assess temperature effects for complicated shotcrete mixtures. As modern binders tend to substitute more and more Portland cement clinker for SCM in order to save money and carbon dioxide emissions, it becomes increasingly important to know the temperature sensitivity of the actual mixture of clinker—SCM—admixture rather than relying on tabulated data for Portland cement.

The temperature dependency of the hydration rate of Portland cement can be described by Arrhenius law. See Equation (1)

$$k = Ze^{-E_a/RT} \quad (1)$$

where k is the rate constant for the reaction, and Z is a proportionality constant that varies from one reaction to another. E_a is the activation energy for the reaction, which in case of most cementitious systems is the so-called “apparent” activation energy, since several complex chemical reactions may take place during the hardening process. R is the ideal gas constant in Joules per mole Kelvin, and T is the temperature in °Kelvin. According to this equation, a plot of $\ln k$ versus $1/T$ gives a straight line with a slope of $-E_a/R$. The calculation of the apparent activation energy is based on the assumption that the overall reaction proceeds along the same path at different temperatures according to the Arrhenius law.

Anderberg & Wasdo (2007) described a simple approach to calculate an apparent activation energy of cementitious mixtures by measuring the heat flow rate in cementitious mixtures tested at multiple temperatures in an isothermal conduction calorimeter. It was assumed that the activation energy is only dependent on the produced heat (the degree of hydration), and therefore the activation energy can be evaluated at each heat (extent of reaction) from the slope of the logarithm of the thermal power vs. the inverse of the absolute temperature. As the thermal power is proportional to the rate of reaction this gives the activation energy without knowledge of the actual rate of reaction.

Figures 5–6 show the effect of temperature on the heat flow rate (power) and the produced energy (approximation for the overall degree of hydration) for a Portland cement paste without admixture, $w/c = 0.45$, made using the same cement as tested with admixture and shotcrete accelerator as shown in Figure 2. Note that the initial non-isothermal heat flow is excluded from the energy calculation in Figure 6. Figure 7 shows the corresponding Arrhenius plot with the slope of a fitted straight line representing the apparent activation energy.

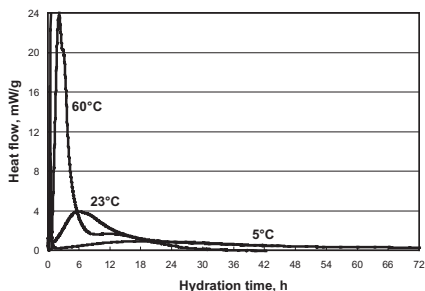


Figure 5. Effect of curing temperature on reaction rate (heat flow rate) for an Australian GP cement tested isothermally as cement paste w/c 0.45 at 5, 23 and 60°C.

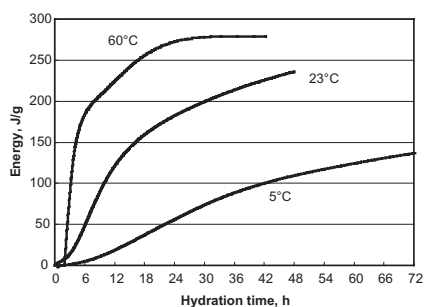


Figure 6. Effect of curing temperature on degree of hydration (energy) for the cement paste shown in Figure 5.

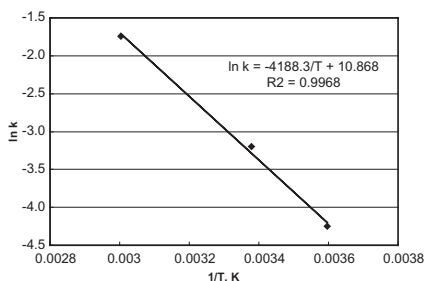


Figure 7. Arrhenius plot based on Equation (1). The slope is the apparent activation energy for the cement hydration process.

The activation energy concept as described using isothermal calorimetry was applied to the Australian GP cement that responded well to an alkali-free shotcrete accelerator, Figure 3, and also to the Australian GP cement that did not respond so well to the same admixtures, Figure 4, using the same mortar method as described. All mortar mixtures were prepared at room temperature and parallel 100 g samples were

put into the isothermal calorimeters set at 5, 23, and 60°C immediately after mixing. Samples were briefly taken out of the calorimeters for addition of shotcrete accelerator 30 minutes after initial mixing, followed by a 30 second re-mix and immediate reloading of samples into the calorimeters. The resulting apparent activation energies using the method described are shown in Figure 8. It seems that the responsiveness of the cement to the accelerator is reflected by a stronger ability to lower the apparent activation energy.

The Arrhenius law (Eqn. 1) applies only if a change in temperature merely changes the speed of reaction and not the major path of reaction. This is usually the case for straight Portland cement mixtures, which in turns forms the foundation for the use of Maturity models for predicting degree of hydration and strength development (Schindler, 2004). However, a “major” change in reaction path going from one temperature to another would invalidate this concept. Cement-admixture incompatibility is sometimes induced by a temperature change such that the major path of cement reaction has significantly changed and therefore may not be detected using standard laboratory room temperature tests (Cost, 2006). The reaction between calcium sulfate and tricalcium aluminate is an example of this, since the reactivity of tricalcium aluminate increases with temperature while the solubility of calcium sulfate decreases with temperature. As a result a cementitious mixture can be at sulfate balance at room temperature but not at elevated temperatures, causing excessive aluminate hydration to poison the strength building Alite hydration at elevated temperature. Since most shotcrete accelerators depend on both temperature and cement chemistry for performance, isothermal calorimetry is an excellent tool for both mixture design optimization and trouble shooting of non performing mixtures.

2.3 Semi-adiabatic calorimetry

Figure 9 shows a schematic design of a simple semi-adiabatic “temperature rise” calorimeter.

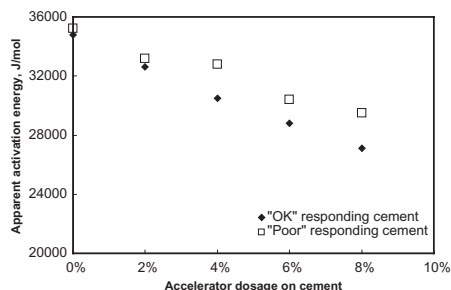


Figure 8. Apparent activation energy by isothermal calorimetry tests of mortar at 5, 23 and 60°C using the methodology described above after Anderberg & Wasdo (2007).

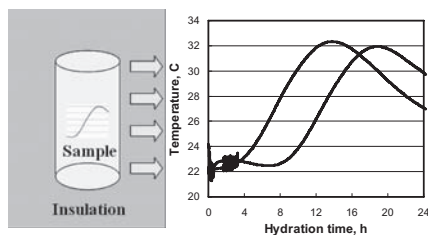


Figure 9. Schematics of a semi-adiabatic calorimeter.

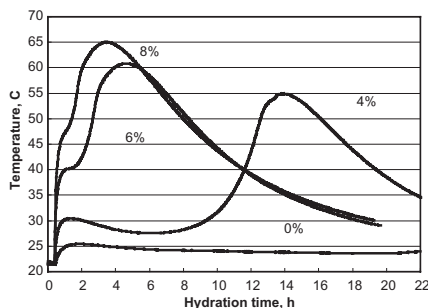


Figure 10. Semi-adiabatic testing of the same mixtures as shown tested isothermally in Figure 2. All mixtures had a fixed w/c and dose of stabilizer and water reducer according to field experience. Accelerator dosed by weight of cement as indicated.

The heat generated from the reacting sample increases the temperature of the sample and causes a temperature-induced acceleration of the cement hydration. As some heat is allowed to dissipate through the insulation, the sample reaches a maximum temperature after reaching its maximum rate of reaction and then cools off, resulting in a temperature curve of somewhat similar shape as an isothermal heat flow curve, Figure 2.

The temperature profile measured by semi-adiabatic calorimetry is similar to that developed using isothermal calorimetry. It is used to evaluate the overall hydration performance of cementitious mixtures with special emphasis on the timing and the size of the main hydration peak that strongly affects setting and early strength development of cementitious mixtures. Semi-adiabatic calorimeters typically measure a temperature rise in an insulated sample without direct control of the temperature of the sample during testing, thereby allowing for low cost while losing some of the repeatability typically associated with isothermal testing. Cost (2006) showed that simple semi-adiabatic temperature profile measurements in cementitious materials can be used to predict abnormal setting and poor strength development and investigate the causes thereof.

Figure 10 shows the result from semi-adiabatic calorimetry testing of the same mortar mixtures as shown tested by isothermal calorimetry in Figure 2. An Australian General Purpose cement was tested in standard ASTM C109 mortar with a fixed dose of stabilizer and water reducer, and a variable dose as indicated of low alkali accelerator dosed 30 minutes after initial mixing. Note that the mixture with 2% accelerator was not tested in the semi-adiabatic calorimeter.

The semi-adiabatic calorimeter yields somewhat similar results to the significantly more expensive isothermal calorimeter, although with less accuracy and detail. The semi-adiabatic calorimeter is therefore ideal for screening of admixture response and assessing the effect of changing materials in a large number of tests in the field, while the isothermal calorimeter, because of its higher level of complexity and cost, is better suited for more advanced testing in a laboratory.

3 CONCLUSIONS

Calorimetric methods have evolved as relatively simple and low cost methods for measuring the rate of cement hydration that is responsible for set and strength development of concrete. The application of calorimetry to the optimization and trouble shooting of shotcrete mixtures shows great potential and is expected to become even more useful as supplementary cementitious materials continue to replace Portland cement clinker in an effort to reduce CO₂ emissions and cost.

REFERENCES

- Anderberg, A. & Wadso, L. 2007. "Drying and Hydration of Cement Based Self-Leveling Flooring Compounds." *Drying Technology*, 25: 12.
- Cost, T. 2006. "Incompatibility of common concrete materials—influential factors, effects, and prevention." Proceedings of the *Concrete Bridge Conference* held May 7–10, in Reno, Nevada.
- DiNoia, T.P. & Sandberg, P.J. 2004. "Alkali-free shotcrete accelerator interactions with cement and admixtures." In Proceedings *Engineering Developments in Shotcrete 2004* conference, Taylor & Francis.
- Sandberg, P. & Roberts, L. 2005. "Cement-admixture interactions related to aluminate control." *J. of ASTM International*, Vol. 2, No. 6.
- Schindler, A.K. 2004. "Effect of temperature on hydration of cementitious materials." *ACI Materials Journal*, Vol. 101, No. 1, pp. 72–81.
- Wadso, L. 2003. "A Multi Channel Isothermal Heat Conduction Calorimeter for Cement Hydration Studies." In *International Congress of the Chemistry of Cement*, Durban, South Africa, 2003.

Factors to consider in using PP fibres in concrete to provide explosive spalling resistance in the event of a fire

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ABSTRACT: The use of polypropylene fibres in concrete to inhibit explosive spalling in the event of a fire is now becoming common practice in many parts of the world, particularly in tunnel construction. The number of projects using this technology is growing and with this has come an increased understanding of the sometimes conflicting technical and practical issues of using fibres to provide explosive spalling resistance. This paper reviews the many factors that must be considered in designing a fibre reinforced concrete, both cast in place and shotcrete, that meets not only the client's requirements for a cost effective construction exhibiting quality, performance and durability, but also the engineer's requirements for assured optimised resistance to explosive spalling. There are also the practical requirements, so important to the contractor, of being able to mix and place the concrete easily, on-time and within the specification. Factors reviewed include fibre types, dosage, performance and safety margins, mixing and distribution, and effect on concrete properties.

1 INTRODUCTION

Concrete has been the primary material for construction for many years and the effect of fire on concrete has long been studied. This has been particularly the case in tunnels, where the impact both on human life and the structural integrity of the construction has always been a major factor in the designer's mind. However, after the Great Belt Tunnel fire in Denmark (1994), the Channel Tunnel fire (1996) (Figure 1) and the Kaprun Tunnel fire in Austria (2000) designers became even more focused on the structural fire protection of concrete tunnel linings. Apart from the tragic loss of human life, the damage sustained to tunnel structures can cause major disruption and enormous financial loss through the suspension of tunnel use and high repair costs. Reports suggest that the financial cost from direct damage and lost revenue of the Channel Tunnel fire was in itself in the order of £200 m.

Investigations into the first Channel Tunnel fire in 1996 (Kirkland, 2002) and the second fire in 2008 (Flynn, 2008) reported that a significant loss of cross sectional area in the tunnel lining had occurred due to severe spalling of the high strength concrete that had been exposed to very high temperatures. Indeed, in some sections, the loss of concrete was sufficiently great that the embedded reinforcement steel had become so exposed that the structural integrity of the tunnel had been placed at risk.



Figure 1. Channel Tunnel fire damage.

This significant damage confirmed that while the strength and durability of high strength concrete is greatly superior to that of conventional concrete mixes (offering improved mechanical properties, low permeability and chemical resistance), it is far more susceptible to fire damage than normal concrete due to its high density and its intolerance to high pore pressures and internal tensile stresses when exposed to high temperatures. The extent of this damage raised significant concerns over the survivability and structural integrity of tunnel linings following a potential fire and prompted a great deal of international research (Kodur, 2000;

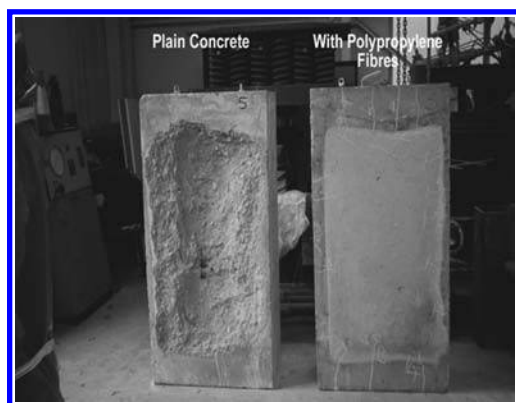


Figure 2. Test samples from CTRL testing—plain concrete & concrete with PP fibres.

Leipzig, 2009) into the factors involved in the spalling of concrete in fires and a search for cost effective solutions.

In the planning for the extension of the high speed rail line from the Channel Tunnel into London, known as the Channel Tunnel Rail Link (CTRL), the designers, following on from experience in refractory products and oil platform protection, commissioned detailed research into concrete containing polypropylene (PP) micro-fibres. Fire testing in small panels (Figure 2) through to full scale tunnel segments clearly demonstrated that the inclusion of appropriate PP monofilament micro-fibres provided the concrete with excellent intrinsic resistance to explosive spalling (Shuttleworth, 2001). The CTRL project subsequently became the first major tunnel project in the world to incorporate PP micro-fibres to provide explosive spalling resistance.

However, not all PP micro-fibres have been found to prevent explosive spalling and with those that do, there can be significant differences in the degree of protection provided or their ability to be used in concrete without changing the mix design. Over the last decade a greater understanding of the detailed requirements needed to be designed into a suitable PP micro-fibre to optimize this technology has been gained. This paper reviews these issues and provides guidance on the specification of PP micro-fibres best suited to fulfil this very important function—the prevention of explosive spalling in concrete.

2 UNDERSTANDING CONCRETE SPALLING

Concrete spalling can be described as the process by which layers or pieces of concrete break off from

the surface of a structural element when exposed to the high and rapidly increasing temperatures experienced in fires (Malhotra, 1984). Three different kinds of concrete spalling can be categorized, as described below.

2.1 Surface spalling

This affects aggregates on the concrete's surface, whereby small pieces of concrete, up to 20 mm in size, are gradually and non-violently dislodged from the surface during the early part of the fire. This is usually caused by the fracture of pieces of aggregate due to physical or chemical change at high temperatures. In the case of surface spalling, the degradation of the concrete is relatively slow and involves the dehydration of the cement matrix followed by the loss of bond between aggregate and matrix. When the temperature rise of the concrete is relatively slow, the moisture in the concrete has time to migrate from the side exposed to the heat and pressure build up is minimal. The presence of moisture in this case can actually mitigate the effects of the temperature rise, since a great deal of energy is consumed in turning moisture to vapour.

2.2 Corner break-off

Corner break-off, which is also known as sloughing-off, occurs at the edges and corners of concrete elements during the latter stages of the fire when the concrete has cracked and weakened.

2.3 Explosive spalling

This is unquestionably the most serious and indeed most dangerous form of spalling that occurs during the first 20–30 minutes of a fire when the temperature in the concrete is in the range of 150–250°C. Explosive spalling occurs when there is a rapid temperature rise, such as in hydrocarbon-fuelled fires following a traffic incident, where very large pieces of concrete can be violently ejected over several metres away from the concrete. As a fresh concrete face is presented to the fire, progressive explosive spalling deep into the concrete thickness occurs, threatening the structural integrity of the construction.

After several decades of research it is known that there is a complex combination of chemical, physical and thermodynamic factors involved that influence explosive spalling. These factors include moisture content, type and size of aggregate, concrete permeability, rate of heating, presence of reinforcement and external loadings. Experts agree that there is a significantly higher risk of explosive spalling when high strength, low permeability

concrete is specified, because of the greater pore pressures that build up during heating.

The theories as to how and why explosive spalling occurs are predominantly based upon moisture movement. As the temperature of the concrete increases, the moisture in the concrete changes to steam vapour. If it is unable to escape from the concrete mass, this vapour creates a dramatic increase in pressure inside the concrete. As this process continues, the vapour pressure increases to the point where it exceeds the tensile capacity of the concrete, causing pieces of concrete to be violently and explosively dislodged from the element. As well as this conventional 'moisture movement' theory, there is also a consensus that aggregate expansion caused by thermal stresses also has a direct influence on explosive spalling.

3 HOW PP FIBRES INHIBIT EXPLOSIVE SPALLING

It has now been well accepted for many years that the addition of suitable polypropylene monofilament micro-fibres (Figure 3) can counteract explosive spalling in cast concrete (Ali et al, 1996) and in shotcrete (Wetzig, 2002), but in order to design an optimized micro-fibre to prevent explosive spalling, it is necessary to have an understanding of the detailed mechanism by which these fibres function. Since the spalling is caused by pressure created by a restriction on the movement of moisture/steam, then somehow the presence of the fibres must relieve that pressure.

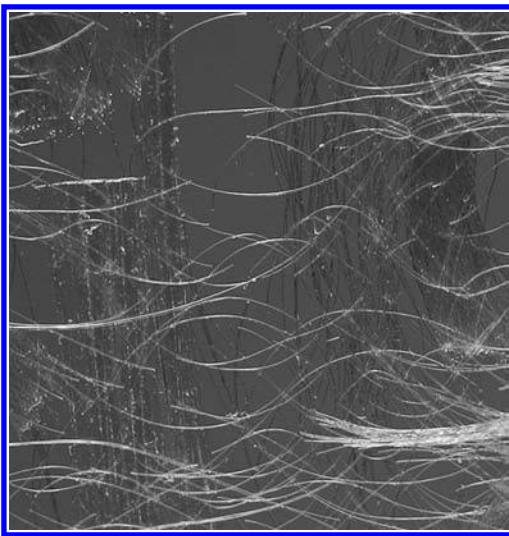


Figure 3. Monofilament polypropylene fibres (PP).

As the temperature in the micro-fibre reinforced concrete rises the PP softens and begins to melt due to a progressive change of phase which starts at approximately 150°C when the crystallinity begins to break down into an amorphous polymer. It peaks at 165°C (the commonly quoted melting point), and is complete at approximately 175°C. It is this melting that is believed to facilitate the reduction in the internal stresses present in the concrete that cause the explosive spalling and there are two main theories that are proposed as to how the micro-fibres do this.

3.1 Mechanisms

Khoury (2008), whilst recognizing the possibility of other mechanisms, advocates what he terms a PITS (Pressure Induced Tangential Space) theory in which the steam overrides the expansion of the PP as it melts, to squeeze between the micro-fibre and the concrete matrix and pass along the length of the fibre. He claims that the effectiveness of such a mechanism would be dependent upon the cumulative surface area of the micro-fibre and connectivity of the fibres, and is therefore favoured by an ultrafine fibre with a diameter of around 18 µm diameter that provides a very high number of fibres. Since micro-fibres are dispersed throughout the concrete as individual entities it is not clear how the connectivity of the fibres is created and how the steam pressure is alleviated. This theory also cannot explain why 32 µm diameter micro-fibres, that provide only one third the number of fibres compared to 18 µm diameter, have been proven to provide comparable and possibly slightly superior explosive spalling resistance (Jansson et al, 2008).

3.2 Microcracking mechanism

An alternative theory was presented by Sullivan (2001) who contends that as an individual PP fibre melts, its much higher coefficient of thermal expansion compared to that of concrete (8.5×) creates a large number of microcracks. These newly created microcracks can then link with the microcracks created by the thermal expansion of neighbouring micro-fibres, or from thermally induced stresses, to form an interconnecting network that can facilitate the movement of steam through the concrete. It is this permeability, which most importantly is created only should a fire event occur, that relieves the stresses created by the steam generation and counteracts the possibility of explosive spalling.

Liu et al, (2008) found from backscattering electron microscopy (BSE) and gas permeability testing that the melting of the PP fibres increased the connectivity of the isolated pores leading to an

increase in permeability, with peak permeability occurring at approximately 200°C (i.e. soon after the melting point of the polypropylene). It was concluded that the creation of microcracks and their connectivity into a network (Figure 4) were major factors in determining the permeability of the concrete upon exposure to high temperatures.

Saka et al, (2009) report from preliminary numerical simulation studies that a single PP fibre embedded in a mortar matrix subjected to a temperature increase of 140°C creates a significant stress on the matrix because of the difference in the coefficients of thermal expansion. Khoury (2008) also indicates a significant tensile stress is exerted on the surrounding matrix by the large difference in thermal expansion between concrete and PP polymer leading to the creation of microcracks.

Microcracks are also created in concrete by thermal effects such as aggregate expansion, drying shrinkage and steam generation. However, it is the supplementary creation of microcracks provided by the melting of the PP micro-fibres that operates in a serial/parallel system with these pores and interfacial transition zones (Kalifa et al, 2001) that provides the superior level of protection of the concrete against explosive spalling.

The larger the mass of the individual fibre, the higher will be the stress that the molten polymer can create and the increased tendency for microcracks to be formed as a result of this stress. However, too large an individual fibre leads to a lower number of fibres distributed throughout the concrete which reduces the network formation possibilities. Equally, at the other extreme, too small an individual fibre reduces the tendency for microcracks to be formed, restricting the network that can be created which reduces the overall ability of

the fibre to prevent explosive spalling (Bostrom & Jansson, 2007). The optimum size of the fibre for the most efficient explosive spalling resistance lies between these two extremes.

3.3 Optimum fibre dimensions

Work done by Jansson & Boström (2008) compared the performance of 12 mm long PP micro-fibres of two different diameters (32 µm and 18 µm) in test panels made with concrete typically used in the construction of tunnels in Sweden. The tests were conducted under conditions designed to encourage an explosive spalling tendency, such as: using the more severe Rijkswaterstatt (RWS) fire curve of 2 hours with a peak temperature of 1350°C versus the standard Eurocode 1 fire curve of 1100°C for 2 hours (EFNARC, 2006) using large panels measuring 1200 × 1700 × 300 mm instead of small panels measuring 500 × 600 × 300 mm, using large aggregate versus smaller aggregate, and testing panels with high moisture content under compressive load and with low fibre dosages. They reported the spalling data shown in Figure 5.

The data shown in Figure 5 obviously invalidates the theory that it is simply the number of fibres in the concrete that determines the effectiveness of a fibre to provide explosive spalling resistance. This is because the 32 µm diameter PP fibres are seen to provide at least comparable performance to 18 µm diameter fibres that are 3.2 times more numerous in the concrete. The data also indicates that panels containing 1.0 kg/m³ of 32 µm diameter fibre gave marginally better results than those containing the higher 1.5 kg/m³ dosage of 18 µm diameter fibre. The number of fibres in the

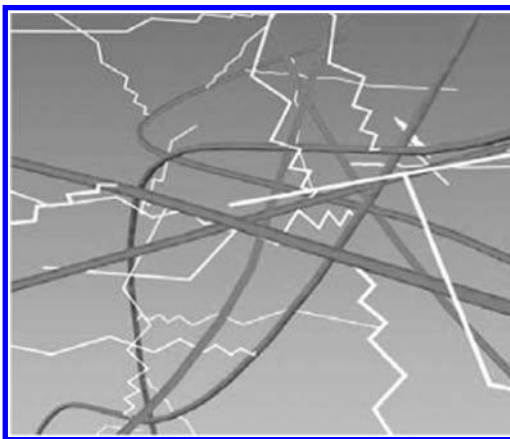


Figure 4. Microcracking network generated in heated concrete.

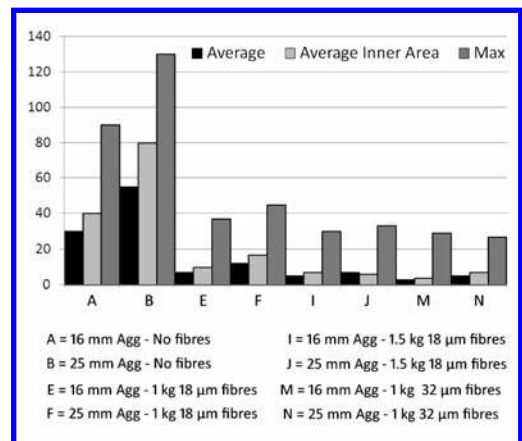


Figure 5. Spalling depths (vertical axis, in mm) for large scale slabs after 30 minute exposure to the RWS fire curve.

concrete is a factor in performance but it is clearly not the dominant factor.

The above referenced research together with in-house research at Propex Concrete Systems in cast concrete and shotcrete (Tatnall, 2002) leads us to support the view put forward by Sullivan and others that it is the expansion of the molten PP fibre, which induces the microcracks to create a network for the pressure relief of the steam, that is the dominant mechanism in providing the explosive spalling resistance in concrete. The essential pre-requisite is that the microcracks must be created before the pressure relieving benefits of any network can be utilised. That mechanism is favoured by using a 32 μm diameter fibre rather than a smaller diameter fibre, such as an 18 μm diameter fibre.

In the same way, a fibre of 12 mm length will increase the individual fibre volume and promote the creation of microcracks in a fibre more than a 6 mm long fibre, whilst still providing sufficient numbers of fibres (approx 120 million/kg) to create the required network for the dissipation of steam vapour. Although a 6 mm fibre can perform to a high level, a 12 mm fibre will be more effective, particularly under more severe conditions when the highest level of performance is required and where differences will be seen between fibres. Besides using a fibre of 32 μm diameter and 12 mm length that favours explosive spalling resistance, there are several other very important requirements for fibres incorporated into concrete that must also be addressed in order to arrive at a practical, viable fibre for the prevention of explosive spalling.

4 GENERAL REQUIREMENTS

With respect to the use of fibres, the different parties involved in a project can have differing requirements. The client wants a fire resistant, durable structure, fast construction, minimum maintenance in service, minimum loss of service during repair, reduced insurance premiums, and a cost effective solution. The designer/engineer wants certified explosive spalling resistance capability, quality assured materials, no negative impact on other concrete properties, ease of use in construction, and a cost effective solution. The contractor/concrete supplier wants a cost effective solution, ease of addition to concrete, and no concerns relating to mixing and distribution in concrete. It is obvious that the best overall specification will be a fibre that provides the optimum balance between proven explosive spalling resistance, practicality (trouble free usage) and cost effectiveness—a fibre that satisfies all of the interested parties requirements.

PP micro-fibres are compatible with steel fibres and chemical admixtures and have been found to

mix, distribute, pump and be cast/wet sprayed in a similar way to unreinforced concrete/shotcrete. It has been found that the fine nature of PP fibres is not compatible with the dry shotcrete system (Wetzig, 2002).

5 TECHNICAL CONSIDERATIONS

5.1 Fibre quality

To provide confidence that the micro-fibres will consistently provide the full range of benefits, all fibres used for explosive spalling resistance should be PP monofilament micro-fibres (100 percent virgin polypropylene fibres containing no reprocessed olefin materials) conforming to EN 14889-2:2006 Class 1a and specifically engineered & manufactured in an ISO 9001 certified facility for use as concrete secondary reinforcement. Where applicable, fibres should also carry the CE marking. Fibrillated PP fibres provide a limited degree of protection whilst macro-synthetic and steel fibres have been found to have little or no influence on the prevention of explosive spalling.

5.2 Fibre dosage

Many researchers have demonstrated that the degree of spalling is influenced not only by fibre type but also by the fibre dosage (e.g. Zeiml et al, 2006, Bilodeau et al, 2004). The concrete specification and the fire risk assessment are important factors to consider when selecting the dosage of PP micro-fibres to use for passive fire protection. Road tunnels generally present greater fire risks than rail tunnels owing to the unpredictability of the vehicles and the nature of goods transported within.

Accurate determination of the actual minimum fibre dosage that will provide the required explosive spalling resistance can, in reality, only be established by large scale fire testing of the actual concrete which is to be used on a specific project. This is a costly exercise and an expense many projects would like to avoid. Section 6.1 of the European Standard EN 1992 Eurocode 2 makes reference to the use of 2 kg/m³ of monofilament polypropylene micro-fibres to control explosive spalling in high strength concrete. Many engineers follow this recommendation to eliminate the need for expensive testing in the knowledge that this dosage will provide a very good safety margin. This does not preclude the use of lower dosages but it does highlight the need for careful consideration and a necessity to carry out fire testing on large concrete samples that completely replicate the materials to be used on an

actual project. Where this has been done, dosage rates of, for example, 1.0 kg/m^3 and 1.5 kg/m^3 have been used in actual tunnel projects.

Whilst small scale sample testing gives indicative performance data that may be useful in making an initial selection of materials, it does not replicate the situation in a real tunnel fire and cannot provide the quality or range of data provided by large scale sample testing (Figure 6). Engineers should also be clear on what they consider to be the acceptable performance for spalling resistance. Some project specifications have stated that during fire testing there will be no explosive spalling permitted. Whilst this is entirely possible in small scale samples and with favourable mix designs, this would be almost impossible to achieve in large scale fire tests conducted on high strength concrete and adopting the most onerous RWS fire curve. What is most important is that the concrete does not experience “progressive spalling” which ultimately puts the entire structure at risk.

If conventional rebar or fabric is to be used in the concrete structure then this should also be included in test samples, as the presence of steel reinforcement will have an influence on the degree of spalling experienced in a fire scenario. It should be noted that the use of PP micro-fibres does not reduce temperature development through the concrete, so careful consideration should be given to the depth of cover requirement over the steel reinforcement.

6 PRACTICAL CONSIDERATIONS

There have been several cases of projects where fibres have been selected purely on spalling results from small scale laboratory testing and then, when full scale site production has begun, the engineers and contractors have seen that some fibre products have a dramatically negative influence on the concrete,

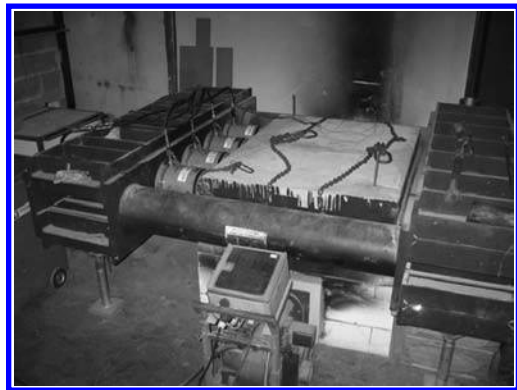


Figure 6. Large panel fire testing.

notably the workability, air content and compressive strength. Changes have then been made to the mix design in order to offset these negative effects and the fire testing data has been effectively rendered null and void. Therefore, during the selection process for PP micro-fibres it is imperative that designers take into consideration the effect of the fibres on concrete workability, air content, and strength.

6.1 *Effect on workability*

The efficacy of all fibre reinforcement is dependent upon achievement of a uniform distribution of the fibres in the concrete, their interaction with the cement matrix, and the ability of the concrete to be successfully cast or sprayed. Essentially, each individual fibre needs to be coated with cement paste to provide any benefit in the concrete. Regular users of fibre reinforcement concrete will fully appreciate that adding more fibres into the concrete, particularly of a very small diameter, results in a progressively more negative effect on workability and the necessity for mix design changes. This is because very small diameter fibres have a much higher combined surface area (e.g. $18 \mu\text{m}$ diameter fibres have a 77% higher surface area than $32 \mu\text{m}$ diameter fibres). This extra demand on the cement paste, unless adjusted for by the addition of more water and cement or admixtures (thereby increasing costs), will ultimately have a dramatic effect on the workability of the concrete, particularly when dosages are above 1 kg/m^3 . Kompen (2008) has reported the experience in Norway that ultrafine fibres in wet shotcrete had an adverse effect on the water demand of the mix and that the fibres were dispersed from the shotcrete during spraying and blocked air filters on the spraying machines.

6.2 *Effect on air content*

Another practical aspect to consider in the selection of PP micro-fibres is that bundles of very small diameter fibres are more difficult to distribute throughout the concrete and are known to entrain more air in the concrete. Comparative site studies have identified that the air content for concrete containing $18 \mu\text{m}$ diameter fibres was around 5–8% compared to approx 1.0% for a $32 \mu\text{m}$ diameter fibre. This increase has a negative effect on concrete strength which is not desirable in underground constructions. Some projects that have used $18 \mu\text{m}$ diameter fibres have even resorted to using de-foaming agents to reduce air content. This will inevitably influence the in-place costs of the concrete and place a question mark over the validity of any fire tests carried out to assess the explosive spalling performance of the original concrete/fibre combination.

With a ~6% reduction in compressive strength for every 1% increase in air content, the use of ultrafine micro-fibres can result in a very significant loss of concrete strength. In an effort to divert attention away from the negative effects these very fine diameter fibres have on workability and air content, lower dosages have been suggested as the solution. Whilst this may be perceived as an interesting commercial proposition for contractors and ready-mixed concrete suppliers whose priority is to have a low cost solution, this suggestion reduces the degree of explosive spalling resistance and should not be accepted on the basis of small panel laboratory testing of concrete having an elevated air content. This increase in air content may not be a problem for shotcrete as the air is blown out during spraying, but if the performance of a fibre for a shotcrete mix is assessed on a panel made with cast concrete, the elevated air content can mask the true ability of the fibre to counteract explosive spalling when it is used in the actual shotcrete because the entrained air improves the possibility of a concrete panel passing a fire test.

6.3 Addition & mixing of PP fibres

The addition of PP monofilament micro-fibres to concrete is a relatively simple process depending on the size of the project. Fibres designed specifically for concrete reinforcement are normally supplied in fully degradable paper packaging that enables the desired dosage per unit volume to be simply added directly into the concrete truck or pan mixer. The packaging is designed to rapidly break down allowing uniform distribution of the fibres into the concrete. In relatively small projects this is often the most cost effective method to adopt, with packaging available in 1 kg or 2 kg bags.

Where projects involve significant quantities of PP micro-fibres, the contractors and ready-mixed suppliers often consider the use of more sophisticated, automated methods of adding fibres to the concrete. Fibre dosing machines ensure that the required amount of fibres is accurately measured and delivered automatically to the concrete mixer. There is a risk when using very fine fibres that because of the extremely high number of fibres being added to the concrete that they may stick together in bundles, becoming encased in cement paste and not evenly distributed throughout the concrete. This has been seen during fibre wash out tests when clumps of fibres have been observed. Obviously this is unacceptable because the protection against explosive spalling is not uniformly distributed in the concrete. This has not been an issue with the 32 μm diameter fibres which have also been found to work very well in the automated fibre dispensing and delivery systems.

6.4 Improvements in shotcrete

The benefits of using suitable PP micro-fibres in shotcrete are not only restricted to enhancing resistance to spalling during fires. PP micro-fibres have been added to wet shotcrete for many years to provide several operational and performance benefits including: the ability to apply greater thicknesses in a single pass, reduced rebound and sloughing, enhanced impact and abrasion properties, early-age crack control, reduced line pressure in spray pumps, reduced dosage of accelerator and increased resistance to freeze-thaw cycling.

7 COST EFFECTIVENESS

The introduction of PP micro-fibres is now widely recognized for providing passive fire protection to both shotcrete (Larsson, 2006) and cast concrete (Court, 2003) structures and an extremely cost effective solution when compared to alternatives such as sprayed coatings or barrier methods. Savings in materials, labour and project time have all been seen in practice. Many major tunneling projects now incorporate this technology to provide insurance against explosive spalling in the event of fire, including for example the Gotthard Base Tunnel (Switzerland), the Brenner Eisenbahn Tunnels (Austria), the Channel Tunnel Rail Link (United Kingdom) (Court, 2003), the Weehawken Tunnel (USA) (Ozdemir, 2006), the Epping-Chatswood Rail Line (Sydney, Australia) (Schmidt, 2005), EastLink Tunnels (Melbourne, Australia) (Highway Engineering, 2007) and the Lane Cove Tunnel (Sydney, Australia). Many clients follow the recommendations of European Standard EN 1992 Eurocode 2 and specify 2 kg/m³ dosage because it provides proven performance with a good safety margin. Depending upon tunnel usage and risk assessment it may be possible to reduce the dosage to a lower level but this may require a programme of large scale fire testing to provide evidence that this dosage is indeed acceptable in a specified mix design. The additional cost for carrying out such testing may well prove unattractive to many projects.

During the selection process, designers and contractors should not simply look at the basic cost of fibres, they should carefully compare “in-place” concrete costs. This will ensure that the positive and negative effects of all fibres, together with any additional cement or admixture costs, are taken into account. Because concrete incorporating 32 μm diameter fibres does not suffer from as many negative side effects as observed when using 18 μm diameter fibres, the former type of PP micro-fibre provides a more cost-effective option.

8 CONCLUSION

If fibres are to be used to provide explosive spalling resistance in shotcrete or cast-in-place concrete they should satisfy two main criteria in order to meet all of the requirements of the interested parties. These include a demonstrated ability to counteract explosive spalling as well as producing no negative side effects in the concrete. These criteria have been found to be best achieved by using a fibre made of polypropylene monofilament of 32 μm diameter and 12 mm length manufactured to, and complying with, ISO 9001 & EN14889-2 standards.

REFERENCES

- Ali, F.A., Connolly, R. & Sullivan, P.J.E., 1996, "Spalling of High Strength Concrete at Elevated Temperatures", *J. Applied Fire Science*, Vol. 6(1) pp. 3–14.
- Bilodeau, A., Kodur, V.K.R. & Hoff, G.C., 2004, "Optimization of the type and amount of polypropylene fibres for preventing the spalling of lightweight concrete subjected to hydrocarbon fire", *Cement and Concrete Composites* Volume 26, Issue 2, February, pp. 163–174.
- Bostrom, L. & Jansson, R., 2007, "Fire Spalling of Self Compacting Concrete", *5th International Symposium on Self Compacting Concrete*, Ghent, Belgium, September.
- British Standards Institution, 2004, BS EN 1992-1-1. Eurocode 2: Design of concrete structure. General rules and rules for buildings.
- British Standards Institution, 2006, BS EN 14889-2. Fibres for concrete. Part 2—Polymer fibres. Definitions, specifications and conformity.
- Court, D., 2003, "Channel Tunnel Rail Link", *Concrete*, October.
- EFNARC, 2006, "Specification and Guidelines for Testing of Passive Fire Protection for Concrete Tunnels Linings" March.
- Flynn, S., 2008, "Fire Stripped off Channel Tunnel Lining", *New Civil Engineer*, 9 October.
- Highway Engineering in Australia, 2007, June, page 8.
- Jansson, R. & Boström, L., 2008, "Experimental Study of the Influence of Polypropylene Fibres on Material Properties and Fire Spalling of Concrete", *3rd International Symposium on Tunnel Safety and Security (ISTSS)*, Stockholm, Sweden.
- Kalifa, P., Chene, G. & Galle, C., 2001, "High Temperature Behaviour of polypropylene fibres", *Cement & Concrete Research*, Vol. 31, Issue 10, October, pp. 1487–1499.
- Khoury, G.A., 2008, "Polypropylene fibres in heated concrete", *Magazine of Concrete Research*, 60(3), April, pp. 189–204.
- Kirkland, C., 2002, "The Fire in the Channel Tunnel", ITA AITES 28th World Congress, Sydney, March.
- Kodur, V.K.R., 2000, "Spalling in High Strength Concrete Exposed to Fire: Concerns, Causes, Critical Parameters and Cures", *ASCE Proceedings of Structures Congress, Advanced Technology in Structural Engineering*, section 48, chapter 1.
- Kompen, R., 2008, "How the Use of Fibres has Developed in Norway", *5th International Symposium on Sprayed Concrete*, Lillehammer, Norway, page 245, April.
- Larsson, K., 2006, "Fires in tunnels and their effect on rock", Research Report, Luleå University of Technology, page 31, February.
- Leipzig, 2009, 1st International Workshop on Concrete Spalling due to Fire Exposure, Leipzig, Germany.
- Liu, X., Yeb, G., De Schutter, G., Yuana, Y. & Taerwe, L., 2008, "On the mechanism of polypropylene fibres in preventing fire spalling in self-compacting and high-performance cement paste", *Cement & Concrete Research*, Vol. 38, Issue 4, pp. 487–499.
- Malhotra, H.L., 1984, "Spalling of Concrete in Fires", CIRIA Technical Note 118.
- Ozdemir, L., 2006, North American Tunneling, Weehawken Tunnel, page 412.
- Saka, T., 2009, "Spalling Potential of fire exposed structural concrete", *Proceedings of 1st International Workshop on Concrete Spalling due to Fire Exposure*, Leipzig, Germany pp. 510–518.
- Schmidt, A., 2005, "Polypropylene Fibres Protect against Explosive Spalling in Sydney Underground", *Concrete Engineering International*, Spring, page 52.
- Shuttleworth, P., 2001, "Fire protection of precast concrete tunnel linings on the Channel Tunnel Rail Link", *Concrete*, April, pp. 38–39.
- Sullivan, P.J.E., 2001, "Deterioration and spalling of high strength concrete under fire", Report for UK Health & Safety Executive, City University London.
- Tatnall, 2002, "Shotcrete in Fires, Effect of Fibers on Explosive Spalling", *Shotcrete*, Fall.
- Wetzig, V., 2002, "The Fire Resistance of Various Types or Air-Placed Concrete", *4th International Symposium on Sprayed Concrete*, page 352, Davos, Switzerland, September.
- Zeiml, M., Leithner, D., Lackner, R. & Mang, H.A., 2006, "How do polypropylene fibers improve the spalling behavior of in-situ concrete" *Cement & Concrete Research*, Vol. 36, Issue 5, May, pp. 929–942.

Composite linings: Ground support and waterproofing through the use of a fully bonded membrane

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ABSTRACT: Robust waterproofing of underground structures is one of the most cost-effective ways to enhance safety and function as well as to increase the design life of new and existing structures. An efficient and economical approach to waterproofing is to integrate a waterproofing layer into ground support through the use of a fully-bonded cementitious polymer-based spray-applied membrane that bonds equally successfully to the inner and outer lining and is unaffected by the concrete placement method or reinforcement type. This solution is an alternative to the more traditional waterproofing systems in a number of situations where the traditional system faces particular difficulties or limitations. The fully bonded membrane acts as a shear connector between the two layers and, where it is possible to rely on the primary lining as a permanent solution, establishes a thinner monolithic waterproof structure. This paper will highlight design considerations and present some international case studies that will illustrate the application of this system.

1 INTRODUCTION

Efficient and cost-effective lining and waterproofing solutions are required to satisfy increasingly demanding programs and to create safer and friendlier space underground. Dry tunnels are a pre-requisite in many tunneling applications, and innovative approaches such as fully-bonded spray-applied membranes, are now proven methods worldwide. The benefits of fully-bonded membranes as a direct replacement for traditional sheet membranes within a support system will be discussed later in Section 3.1.

Additional advantages of the system are gained from the sprayed nature of the membrane such as the possibility of using sprayed concrete not only as a primary lining (often considered only as a temporary support) but also as a secondary permanent lining, instead of cast insitu concrete. This has the potential of reducing the thickness of the concrete secondary lining where typically the thickness of a final cast insitu lining is in excess of 250 mm. This figure is often driven by formwork and reinforcement placement tolerances and cover requirements and is not purely based on the design loads. If formwork and reinforcement, in the form of bars, can be eliminated, through the use of Fiber Reinforced Sprayed Concrete (FRSC), the thickness of the inner lining can be reduced.

In addition to this, where it is also possible to rely on the primary layer as a permanent solution, a thinner monolithic waterproof structure is established. Both layers of concrete, primary and



Figure 1. Sample cores of a fully bonded spray applied membrane sandwiched between two sprayed concrete layers.

secondary, can therefore be considered permanent and durable elements that can and will fulfill the structural requirements both during construction and throughout the designed life of the structure. The integration of a state-of-the-art fully bonded cementitious polymer-based spray-applied membrane into concrete linings has been developed for waterproofing and at the same time to provide a shear connection between primary and secondary layers of concrete.

2 THE SYSTEM

This unique composite system consists of a first layer of sprayed concrete (temporary or permanent), a fully bonded intermediate layer of elastic cementitious polymeric spray-applied membrane, typically 3 mm thick, and a second layer of sprayed or cast-in-place concrete.

A number of tunnels, cross-passages, stations and shafts have been successfully completed worldwide with this system over the last few years, under quite different conditions and design requirements, demonstrating its cost-effectiveness and technical versatility. Some examples are reported in this paper.

3 DESIGN CONSIDERATIONS

The traditional ground support method of conventionally excavated tunnels in simplistic terms considers the primary lining as a temporary lining carrying the ground loads and the secondary lining as permanent carrying the hydrostatic loads.

Design and construction methodologies are evolving and continuously adapting to provide the best possible solution to project-specific ground and loading conditions. Over recent years it has been possible to observe the following trends:

1. The substitution of traditional sheet membranes with fully bonded spray applied membranes in situations where traditional sheet membranes are particularly tricky to apply;
2. A shift from temporary to permanent sprayed concrete primary linings;
3. A shift from reinforced cast insitu secondary linings to FRSC secondary linings, with considerable savings, both in terms of programme duration and materials.

Examples of some of the projects that have followed these trends are given later.

3.1 *Substitution of traditional sheet membranes with fully bonded spray applied membranes*

Spray applied membranes are particularly advantageous in geometrically complex areas such as in lay-by niches, cross passages (Fig. 2), turn-outs and crossover caverns, where installation of



Figure 2. Complex intersections on the Heathrow baggage tunnel.

conventional membranes is inherently difficult and testing is challenging.

This type of membrane can be applied to limited sections to provide isolated waterproofing, for instance in the crown of tunnels in a drained “umbrella” type configuration, or as a continuous waterproofing system (fully tanked, undrained) without discrete joints or need for waterstops and compartmentalization.

Spray applied membranes are faster and more easily installed than sheet membrane thus freeing up time and space for other activities. Typically 50–100 m²/h can be manually sprayed by 2–3 operators, whilst robotic spraying can reach up to 180 m²/h, against the standard 25 m²/h for sheet membranes. A spray applied membrane does not require surface evenness however the time needed for the substrate preparation works and the consumption of material will depend upon the substrate smoothness and quality encountered (Fig. 3).

The bond properties between the membrane and concrete lining on both sides, but particularly to the inner lining, makes the interface between membrane and concrete impermeable. Hence, no migration of water along the membrane-concrete interface can occur mitigating the risk of water ingress by eliminating potential groundwater paths. This is a very important property as it makes a bonded spray-applied membrane fundamentally different from other waterproofing systems like sheet membrane systems with drainage geotextiles (Figures 4 and 5). An eventual seepage through the membrane can be easily resolved locally precisely where the seepage occurs since this point corresponds to the seepage channel in the concrete behind the membrane. For a leak to occur in a double bonded system, water has to find a path through three failure zones. Firstly a crack in the primary lining, then a tear in the membrane followed by a second crack in the secondary lining. All of these failure zones need to occur in the same spot as water cannot migrate along the membrane-concrete interface surface.

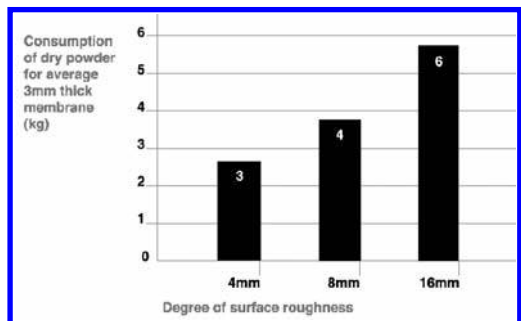


Figure 3. Consumption rates of Masterseal® 345 for various degrees of surface roughness (maximum aggregate size).

Evidence of the bond strength and crack bridging properties of the fully bonded membrane is given in sections 4.1.2 and 4.1.4.

Fully-bonded spray-applied membranes are compatible with other waterproofing systems, i.e. sheet membranes, and standard joint details between and spray applied and sheet membranes have been established, making the system totally flexible (Fig. 6).

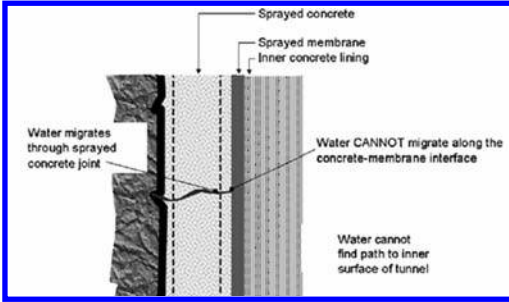


Figure 4. Fully bonded Masterseal® 345 waterproofing membrane preventing the migration of water along the concrete-membrane interface.

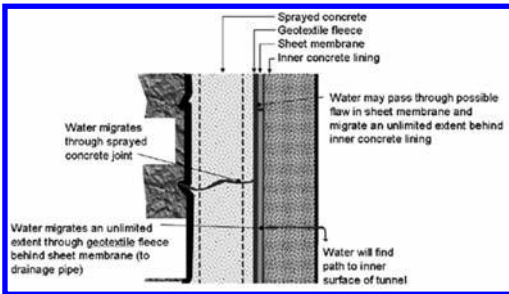


Figure 5. Water migration along the geotextile layer or between the sheet and the inner lining in a traditional support and waterproofing system.

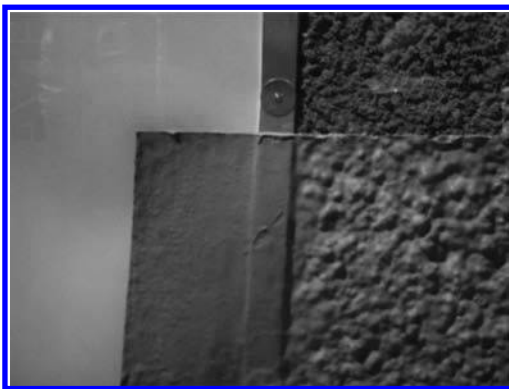


Figure 6. Interface detail between traditional type waterproofing and fully bonded spray applied membranes.

They are also compatible with:

- All concrete placement techniques, allowing placement of a sprayed inner lining which reduces the overall final lining thickness;
- Reinforcement types, mesh, reinforcement bars and fibers, on either side of the membrane, refer to section 3.3;
- Steel insertions (e.g. anchored reinforcement, drainage pipes, etc.).

The spray applied membrane is innately more robust than a sheet membrane solution, in that the creasing, folding, stretching and buoyancy effects, which are unseen but inevitable as wet concrete rises up the sidewalls of an arched tunnel profile, are avoided.

3.2 Shift from temporary to permanent sprayed concrete primary linings

It is important to note that sprayed concrete is only one of several methods to place concrete. The method of spraying concrete onto a surface is ideally suited for the support and construction of underground excavations. The on-going advance in material technology related to sprayed concrete, particularly with the advent of micro- and nano-silica technology, alkali-free accelerators and fiber reinforcement, has created a high level of confidence in attaining a tunnel lining with excellent durability characteristics and watertightness. In addition to materials improvement, the construction process has become highly automated thereby significantly reducing the degree of human influence that has, in the past, contributed to preventing clients from considering sprayed concrete as permanent support. Effort has also been put into the training of personnel and the creation of regulations (e.g. EFNARC nozzleman certification scheme) that have largely improved the quality of the sprayed product.

These two facets have contributed over recent years to a higher quality sprayed concrete. What once was considered an extremely porous material is now a high quality concrete with high density, low permeability and good durability. The quality of sprayed concrete linings has recently been recognized and designers have begun to rely on the primary lining as a permanent solution. Both layers of concrete, primary and secondary, can therefore be considered as durable structural elements. This is one of the key assumptions for composite behavior, refer to Section 5.

3.3 Shift from reinforced cast insitu secondary linings to FRSC secondary linings

Based on the successes mentioned above, permanent reinforced cast insitu concrete linings can be substituted for more economical permanent FRSC linings.

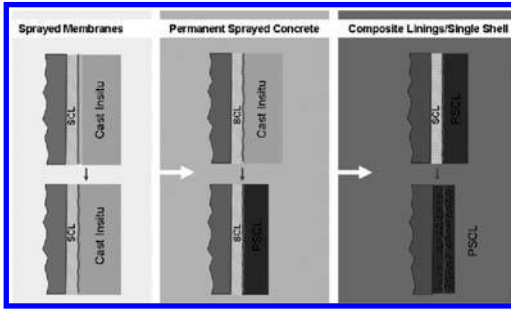


Figure 7. Trends in waterproofing and support systems (SCL = Sprayed Concrete Lining; PSCL = Permanent Sprayed Concrete Lining).

Coupled with a spray applied waterproofing membrane a permanent spray-applied inner lining is a solution which has the potential of reducing the thickness of the concrete secondary lining. Typically the thickness of a final cast insitu lining is in excess of 250 mm. This figure is often driven by formwork and reinforcement placement tolerances and cover requirements, and is not purely based on the design loads. Expensive formwork and reinforcement, in the form of bars, can be eliminated through the use of fiber reinforced sprayed concrete thereby reducing cost and time and the thickness of the inner lining.

4 THE FULLY BONDED WATERPROOF MEMBRANE

4.1 Properties of Masterseal® 345

Laboratory tests in accordance with international testing methods, as listed in Table 1, have been performed on the Masterseal® 345 membrane. The properties of the membrane are described in this section; the interface properties derived from the bond however, will be discussed later in Section 5.

4.1.1 Water pressure resistance and permeability

A tight polymer membrane is in general an excellent waterproofing barrier in its own right. The water flow is prohibited by the strong matrix of polymer chains. Water having a high surface tension is unable to penetrate into the material and pass this barrier.

Masterseal® 345 pores are 20,000 times smaller than a water drop but 700 times bigger than water molecules and therefore water is absorbed by the membrane and released as vapor. Vapor penetrates through the Masterseal® 345 membrane at the same rate as through concrete. The membrane

Table 1. Properties of Masterseal® 345.

Property	Testing method	Minimum specification
Water pressure resistance and permeability	EN 12390-8: 2000 Testing hardened concrete. Depth of penetration of water under pressure	Resist up to 20 Bar to 12 months
Tensile strength at yield	ASTM D638M DIN 53504 Type S2 BS EN 180527-2	From 4 MPa (at +20°C) to 0.6 MPa when submerged
Elongation at yield	ASTM D638M DIN 53504 Type S2	80%–140% (between –20°C and +20°C)
Crack bridging	EN 14224	100% of thickness
Modulus of elasticity	N/A	Variable. See Figure 8 and 9.
Bond strength (both sides)	EN 1542 EN 1766	From 1.0 MPa to 0.5 MPa when submerged
Bulk density	N/A	585 ± 90 g/L ± 100 g/L
Water vapor diffusion	DIN 52615	Max. 300
Resistance factor		
Frost insulation	N/A	5 cm of sprayed concrete the composite lining can resist up to 5000 h °C
Fire resistance	DIN 4102	B2 or better
Rock water resistance	Resistant to Sulfates Resistant to acidic water Resistant to alkaline water	To be specified according to local geological conditions
Potable water certificate	BS 6920	Required when potable water is in direct contact to the membrane

itself should have a nominal thickness of 3 mm (localized minimum of 2 mm can be accepted and maximum of 10 mm) in order to be watertight. Above 10 mm the self-weight of the membrane may cause de-bonding during application.

The watertightness of Masterseal® 345 was successfully tested in BASF laboratories as well as at the independent test institute, EMPA (Federal Institute of Material Testing, Switzerland) and

at BMI—the Technical University of Innsbruck (Austria). Water-tightness tests performed up to 1 year, after which tests were stopped, show that the membrane has excellent resistance to water ingress and can sustain up to 20 bar pressure. It must be noted, however, that the membrane can only withstand active pressure once completely cured. Tests were performed in accordance with EN 12390-8. A layer of 3 mm of Masterseal® 345 were applied to a porous building material of the size 200 × 200 × 55 mm. Samples were stored at 20°C and 65% RH.

Six samples with Masterseal® 345 and one sample without (the reference sample) were glued in pressure vessels so that only the membrane was exposed to the water pressure. Testing was started by flooding the vessels with water and applying a constant pressure of 20 bar. After 1 hour of applied pressure 2.2 L of water flowed through the reference sample and testing was stopped immediately. In the Masterseal® 345 samples however, up to the end of the testing, 3 months and 12 months, the unexposed surfaces of all samples were dry. Three samples were removed from the vessels and split after 3 months and the remaining three samples after 12 months. The penetration depth of water into the test surface was determined on the fresh crack surface immediately after splitting. The reference sample without Masterseal® 345 showed high water inflow, as expected. The samples coated with 3 mm of Masterseal® 345 showed small and homogeneous penetration of moisture from the membrane. The penetration depths into the underlying porous building material after up to 3 months of high pressure application (20 bars) varied between 10 to 22 mm without indication of speeding up over time. In fact the depths of penetration after up to 12 months vary between 6 to 22 mm.

Apart from the observed moisture migration, Masterseal® 345 demonstrates excellent watertight performance characteristics under these adverse conditions. Water migration along the membrane/substrate contact was tested at the University of Innsbruck as the membrane bonds to the surface (following contours). This found that water migration along the membrane/substrate contact is negligible.

4.1.2 Adhesive tensile strength, elongation and crack bridging behavior

Various types of fracture can occur in a composite lining, depending on which part of the system breaks down under the stress of testing:

- Crack in the concrete;
- De-bonding of the sealing membrane (Masterseal® 345);
- Tensile failure of the membrane.

The adhesive tensile properties of the membrane are such that they can:

- prevent debonding of the membrane from the concrete such that there is no expansion of water between these layers and no registered increase in pressure by a manometer;
- bridge cracks and protect the inner shell from being subjected to the stress of water pressure. Masterseal® 345 is able to close and isolate cracks and minimize the surface area over which the water pressure is acting thereby keeping the force acting on the inner lining down to a minimum. This leads to the outer shell easily being able to sustain loads resulting from water pressure consequently justifying the reduction in thickness of the inner lining.

The sealing layer is therefore able not only to seal-off (waterproof) the system, but also to bridge and bond any cracks and redistribute the shearing forces effectively. The tensile strength of the membrane has been tested in accordance with ASTM D628M, DIN 53504 Type S2 and BS EN 180527-2. Results of these tests show that the tensile strength of the cured membrane ranges from 1 to 4 MPa (between -20°C and +20°C). The elongation at yield values ranged from 80% to 140% (between -20°C and +20°C). Thanks to these properties, the membrane provides crack bridging capability some hours after application. Since hardening of the membrane reduces the elasticity, the minimum crack bridging ability is achieved once fully cured. For a 3 mm thick membrane layer this ability is 3 mm. Tests were performed in accordance with EN 14224.

4.1.3 Modulus of elasticity

The stiffness of Masterseal® 345 is not constant, reducing with increasing elongation (Figures 8 and 9). The stiffness values are a function derived from the Tension (Fmax) and Elongation (stress vs strain) curves. These values have been obtained from in-house laboratory tests.

4.1.4 Normal bond strength

The outstanding feature of the membrane is its bond strength. When embedded into concrete it bonds on both sides promoting excellent watertightness, by preventing the development of water migration on both concrete-membrane interfaces, and facilitating shear connection properties.

The bond is provided by the membrane's polymer-rich cement base which prevents de-bonding and ensures the long-term stability of the system. The strong bond is given by crystals produced during the hydration of the cement. These crystals are embedded on the surface of cured concrete and grow into fresh applied membrane paste. The crystals are also embedded in the open exposed surface of the cured

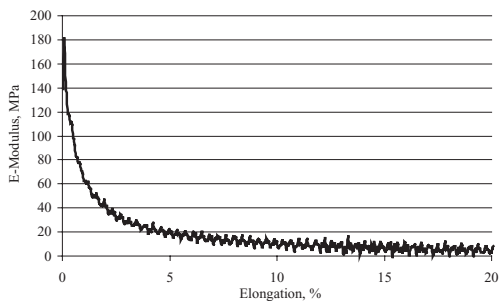


Figure 8. Stiffness values (E -modulus) plotted against elongation in percent of Masterseal® 345 dry membrane.

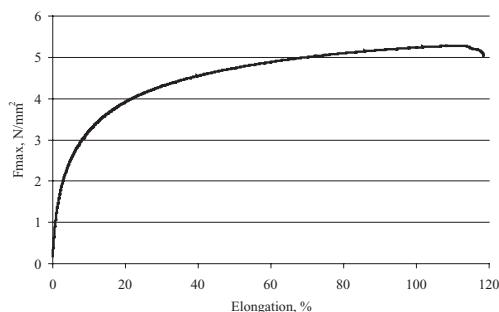


Figure 9. Tensile Force (F_{max}) plotted against elongation in percent of Masterseal® 345.

membrane and grow into fresh applied concrete, providing the chemical and mechanical bond.

The bond is unaltered by the concrete placement technique, be it sprayed or cast insitu, or by the presence of fibers. Tests conducted in accordance with EN 1542 and EN 1766 show that the adhesive (normal) bond strength to clean and particle free cementitious materials is significant and easily reaches $1.2 \text{ MPa} \pm 0.2 \text{ MPa}$ and reduces to 0.5 MPa when water saturated (Figure 10). The bond strength to metals ranges from 0.5 to 1.2 MPa .

4.1.5 Bulk density

Bulk density ($+20^\circ\text{C}$) is $590 \text{ kg/m}^3 \pm 100 \text{ kg/m}^3$.

4.1.6 Water vapor diffusion resistance factor

Water vapor diffusion resistance factor, μ , of max. 300.

4.1.7 Frost insulation

The membrane itself does not have any frost insulating effect. The frost-insulation properties of the composite liner system depend on the thickness of the inner lining of concrete. When freezing can be expected, one needs to establish the design freezing parameter F_{100} in $\text{h}/^\circ\text{C}$, the maximum number of

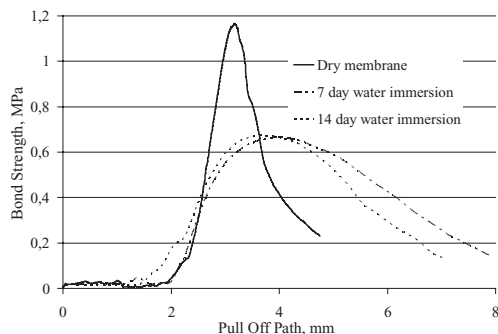


Figure 10. Adhesion of Masterseal® 345 to concrete.

hours of temperature below 0°C in a 100 year period. With an inner lining thickness of 5 cm sprayed concrete, this composite liner can resist up to $5000 \text{ h } ^\circ\text{C}$.

4.1.8 Fire resistance

The membrane does not contain any flammable components, in the event of a fire it is self-extinguishing. When exposed to higher temperatures, approximately 200°C , the membrane softens and then melts emitting carbon dioxide and water (vapor). In the event of a fire, and depending on the intensity of the fire, the damaged area within the tunnel may be refurbished by removing locally the damaged concrete lining and waterproofing. As the membrane melts the bond with the concrete on either side will be lost and restored once the temperature decreases as the membrane hardens once again.

5 COMPOSITE LININGS

The fully bonded nature of the membrane allows the lining to act as a composite structure wherever the primary sprayed concrete can be taken to contribute to the final lining. Great saving can be achieved over the traditional approach, by considering no part of the lining as a “temporary support”, and by the reduction in excavation volume and lining material, which leads to a reduction in construction time.

In a real life situation the two linings behave either as a true “single-shell” or act monolithically through the deformation of the membrane, depending on the magnitude and direction of the applied loads (radial and tangential) and subsequent relative deformation of each lining layer.

The principle of composite action can be demonstrated by comparing the action of two joists placed one on top of the other. If these are simply placed one on top of the other and loaded as a beam there will be some relative movement between the two. However, if these are physically connected, the bending strength and stiffness are

significantly improved as the two will act together as a single unit with combined thickness.

Traditionally, primary and secondary linings could be thought of as acting as a “single-shell” thanks to the shear connection provided by the roughness of the surface. However, an improvement in performance is achieved by connecting the primary concrete lining to the secondary concrete lining by means of a fully bonded membrane that can accommodate the shear forces. As with the so-called “single shell” approach (Golser & Kienberger 1997), it can be assumed the two sprayed concrete layers act together as a composite structure. Aside from the obvious waterproofing advantages, highlighted in section 3.1, a fully bonded membrane is therefore also structurally advantageous.

The capacity of the membrane to allow shear stress transfer across its section allows load sharing between sprayed concrete layers. The composite lining as a whole is able to withstand all potential loading conditions from the ground, ground water and surface loads throughout the design life of the tunnel and at the same time is watertight, durable, capable of accommodating the loads of internal structures such as lighting canopies and ventilation fans and have a surface finish to achieve the required reflectance and aesthetic appearance.

These interface properties are of particular importance and are currently being investigated. Some tests have already been performed, the results of which are discussed in Section 5.1, while other tests are on-going. The excellent bond allows it to be an attractive solution on new as well as on rehabilitation projects, including masonry and brick tunnels, where limited space presents an important issue. Composite linings are a flexible, logical, modern and economical solution.

5.1 Interface properties

To achieve truly composite behavior and guarantee the structural effectiveness of the system, a bond needs to be achieved between the concrete layers and the sprayed membrane to permit the transfer of normal and shear forces between the primary and secondary layers.

The bond strength required at the interface between the primary and secondary lining to permit the composite action required in so-called “single-shell” linings must be evaluated for each project. Studies are being conducted to address the question of the bond (normal and shear) strength required at the interface between the primary and secondary lining to permit the composite action. However, each project must consider the prevailing load conditions before coming to a judgment on whether or not a composite lining solution is achievable.



Figure 11. Masterseal® 345 membrane bridging over a considerable crack.

Relative creep and shrinkage movement of the two layers will cause stresses in the membrane. This may constitute an issue wherever the relative deformations are larger than the membrane’s elongation potential, however, some movement can be taken by the membrane and it is expected that most of the movement in the primary lining will have taken place prior to the installation of the membrane and secondary lining. In order to define the whole system as working as a sandwich of sprayed concrete—waterproofing—sprayed concrete, an in-house research programme has already been initiated.

5.1.1 Shear behavior

When a shearing load is “slowly” applied to the composite structure, thanks to its particular chemistry, the membrane has sufficient time to adapt to the imposed deformation through the rearrangement of its molecules. In this case the membrane behaves in a viscous manner and is not compromised.

When load deformations are applied at higher rates, the composite lining resists with shear strength parameters given in Table 2. These shear strength parameters are taken from direct shear tests carried out under zero normal displacement conditions (constant normal load) to a 2 mm thick membrane applied to a smooth substrate and a 5 mm thick membrane applied to a rough substrate. These two specimen conditions have been taken from typical membrane consumption rates and thickness applications on actual projects and represent two ends of a spectrum, a float finished sprayed concrete substrate coupled with a minimum thickness of membrane and a rougher substrate coupled with a thicker membrane layer. Quantification of roughness is always difficult and in this case was done according to the Joint Roughness Coefficient (JRC) index. The smooth surface of specimen 00 has a JRC of 0 and the rough surface of specimen 03 JCR of 20 (Fig. 12).

Tests were performed once the membrane had been allowed to cure for a minimum of 28 days and

the concrete to dry out (min. 4–6 hours, depending on environmental conditions).

Normal stiffness tests have been carried out by controlling the normal stress. Since, in a tunnel, loading is controlled by the displacement of the ground or existing lining, future tests will be carried out with controlled normal displacement, instead of the normal stress. Normal stiffness insitu depends on the specific application (stiffness of the ground and reinforcement/linings). To overcome this issue, direct shear tests at constant normal stress can give a complete constitutive behavior that includes mobilization of shear strength and dilatancy at each shear displacement.

5.1.2 Bending behavior

In-house Centre Point Load tests, in accordance with EN 12390: Parts 1 and 5, were undertaken on 6 month old (fully cured) specimens having thicknesses of Masterseal® 345 of between 2.5 and 4 mm.

Table 2. Results of direct shear tests.

Parameter	Specimen 00	Specimen 03
Masterseal 345 membrane thickness, mm	2	5
Sprayed concrete roughness	Plane saw cut	Shotcrete roughness
Joint Roughness Coefficient, JRC	0	20
Vertical modulus (first load), MPa	34.5	32
Vertical modulus (unload), MPa	56	40
Shear modulus, MPa	17.5	7.2
Maximum shear stress, MPa	1.01	1.7
Cohesion (c), MPa	0.5	1.05
Friction angle (ϕ), °	24	43

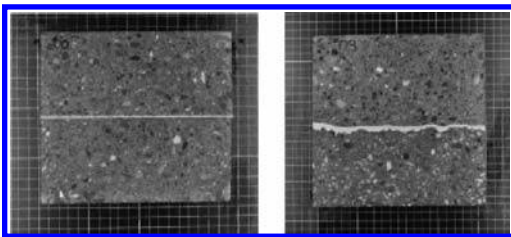


Figure 12. Two specimen blocks for the direct shear test. On the left the specimen with a 2 mm membrane sprayed onto a smooth substrate. On the right a 5 mm membrane sprayed onto a rougher substrate.

Flexural load actions are usually non-uniform, often arising from single-point loads, and involve interactions between several concurrent load actions, such as axial and shear stresses.

The results from these tests indicate the flexural strength and the post-crack performance in terms of residual strength or energy absorption. The specimens were produced by spraying a fiber-reinforced concrete mix with a maximum aggregate particle size of 10 mm. The minimum average flexural strength, f_{cr} , of the specimen without a membrane was 6 MPa and 3 MPa when the membrane is installed centrally with respect to the composite system. It is however evident from the results that, despite the flexural strength decreasing with the integration of a less stiff membrane, the post-cracking or residual flexural strength of a composite beam inclusive of a membrane is higher than that of an equivalent beam without membrane.

5.1.3 Analysis

An investigation of composite behavior was undertaken by Mott MacDonald to simulate composite action and examine the boundaries of its applicability.

The properties of the interfaces between the concrete and the waterproofing membrane were taken from a back-analysis of shear test data gathered at the Technical University of Graz. The original shear test curves were replaced with curves derived from FLAC (Fast LaGrangian Analysis of Continua) simulations of the shear tests. The input parameters for the model are derived by first back-analyzing this test data and calibrating the numerical model. The FLAC base model consisted of a 1 m thick length of composite tunnel lining with 280 mm thick primary lining, a 3 mm layer of Masterseal® 345 and a 145 mm thick secondary lining.

An external pressure was applied to the ring, simulating a vertical stress of 500 kPa and a horizontal stress of 1000 kPa (i.e. the ratio of horizontal to vertical stresses, $K=2$). This represents



Figure 13. Centre Point Load test on a sprayed concrete specimen with a horizontal layer of Masterseal® 345 at mid section.

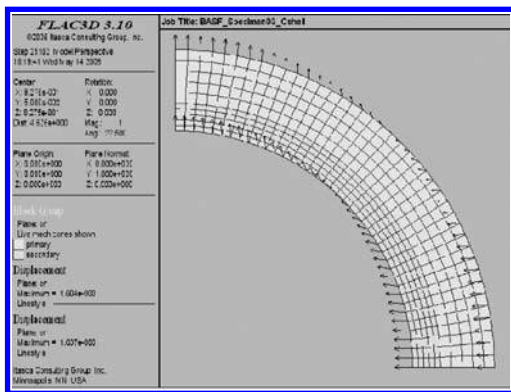


Figure 14. Composite tunnel lining with Masterseal® 345 interface in the FLAC 3D model.

a highly loaded tunnel lining, subjected to bending as well as compression. Most tunnels would experience lower loads than this.

The numerical modeling showed that composite action is possible with Masterseal® 345 and that the performance is relatively insensitive to the precise strength or stiffness values of the bond between the membrane and the concrete.

6 QUALITY CONTROL AND DURABILITY

6.1 Quality control

Sprayed concrete and sprayed membranes, as with any material constructed insitu, suffer the problem that there remain residual concerns about quality control and workmanship. However they have been successfully used on a number of projects. Recommendations for quality control test methods have been made and it is considered that a robust quality control system can be implemented on site. Pre-construction trials and training of operatives are vital for a successful application.

6.2 Durability and waterproofing philosophy

Masterseal® 345 is designed to be spray-applied in a sandwich system between layers of sprayed or cast concrete. This composite system is the key to optimum durability.

For example, in most applications this should limit the temperatures to which the membrane is exposed during its life and prevent exposure to UV radiation. Because this product has been on the market for a relatively short time definitive statements on durability cannot be made. However, this type of polymer has been used in construction for many years and is very stable chemically.

The membrane material consists of Ethylene-vinylacetate (EVA) polymers. In a tunnel environment, the only relevant decomposition mechanism for EVA is by thermal ageing. All materials show some limited rate of change of properties over time, depending on exposure to various conditions and environmental effects. However the chemical nature of polymer-modified mortars in outdoor and indoor exposure was examined by Schulze & Killermann (2000). Over a ten year period, the EVA copolymer gave stable adhesive, flexural and compressive strength. According to scanning electron microscopy (SEM) the morphology of the polymer in the mortars does not change over this period.

However, the long-term durability of a composite lining is more than the chemical durability of the membrane material itself and involves an understanding of which failure mechanisms can potentially occur. An intact and complete sandwich structure is essential to avoid mechanisms which can lead to failure over time. Thus maximization of the structural durability and watertightness of sprayed concrete is possible through the minimization of the potential for water paths through cracks or construction joints, and also by considering the appropriate sprayed concrete mix design to reduce permeability. Water permeability and oxygen diffusion tests from numerous projects have demonstrated the low permeability of sprayed concrete despite the addition of accelerator, super plasticizer and stabilizer/retarder admixtures which are currently essential components of the wet-mix design.

Steel fiber reinforcement has often been used to replace conventional welded steel mesh in permanent sprayed concrete linings to provide additional material safety by reducing shadowing and maintaining early age thermal cracking down to 0.2 mm. Of particular benefit to a lining supporting the ground is the ductile nature imparted by the steel fibers. This allows greater redistribution of loads in the lining, and should the tunnel lining crack, considerable residual strength will exist, thus allowing time to implement contingency actions if necessary.

7 CASE HISTORIES

Composite linings as ground support and waterproofing systems have been adopted as the functional waterproofing and tunnel support on many projects worldwide including the recently constructed Viret Tunnel as part of the Lausanne Metro M2 in Switzerland and the concrete lined Chekka Road Tunnel in Lebanon (rehabilitation project). The design of these tunnel linings comprises both drained and undrained solutions.

Sections of the Hindhead A3 Tunnels and Thames Water's Hampton Shaft, in the UK, have

also been designed using this ground support and waterproofing system. Details of some of these and other projects are discussed below.

7.1 Giswil tunnel

An emergency escape tunnel running parallel to the 1960 m long Giswil road tunnel in central Switzerland was excavated by a 4.0 m diameter hard-rock TBM through mica schists. A length of approximately 10 m at each end of the tunnel was excavated by drill-and-blast. In the portal areas a cut-and-cover concrete tunnel was constructed.

A waterproofing solution was required that provided no water ingress into the bored part of the tunnel, a successful solution for waterproofing continuously from the bored tunnel, through the blasted part of the tunnel to the cut-and cover concrete tunnel at the portal. Compatibility between the waterproofing methods was required. An undrained solution, with a waterproofing system that covered the full tunnel profile, including the invert, was proposed.

The main advantages of proposing a spray applied fully bonded membrane (Masterseal® 345) were: high bond strength to concrete preventing migration of water along the membrane/substrate interface, hence significantly reducing the risk of water seepage even by eventual damage or holes in the membrane; the feasibility of waterproofing the invert; the feasibility of waterproofing the interfaces between bored tunnel and drill-and-blast excavated tunnel, as well as between drill and blast excavated tunnel and concrete cut and cover tunnel; compatibility with the PVC sheet membrane at the interface with the running tunnel; and injection rather than drainage could be performed for removal of water seepages due to the adoption of an undrained solution.

The removal of water seepages around the tunnel profile, which would otherwise cause problems with the bonding of the membrane to the substrate, was done by means of one-component polyurethane injection and two-component acrylic injection (Fig. 15). Manual application of an optional smoothing layer of sprayed concrete was applied to limited sections with a 4 mm maximum grain size to provide a suitable substrate for cost-effective spray membrane application (Fig. 16). Removal of the remaining water seepages through the substrate was implemented by small scale acrylic injection (mainly sealing/filling of dripping bolt holes) (Fig. 17). These injection works were done successfully and with very limited effort. The Masterseal® 345 waterproofing membrane, minimum thickness 3 mm, was sprayed onto the crown and walls (Fig. 18). Frequent spot thickness controls with needles were done during spraying (min-



Figure 15. Temporary removal of water seepages before applying the sprayed concrete substrate (Giswil tunnel).



Figure 16. Application of sprayed concrete to the substrate via MEYCO Oruga manipulator prior to the spraying of the membrane (Giswil tunnel).



Figure 17. Before the spraying of membrane: removal of the last drip spots through rock bolt holes with injections with MEYCO® MP308 (Giswil tunnel).

imum every 2 to 3 minutes). The nozzle man was always kept updated as to the effective thickness which was sprayed. Average capacities of 70–80 m² per hour were achieved.



Figure 18. Set-up for the application of Masterseal® 345 with MEYCO® Piccola dry sprayed concrete machine (Giswil tunnel).



Figure 19. Application of the inner concrete layer with wet-mix sprayed concrete (Giswil tunnel).

Spraying of the unreinforced inner concrete layer was then effected, design thickness 10 cm to the crown and walls. The spraying of the inner concrete took place 1 day after the application of the membrane (Fig. 19). A 150 mm horizontal strip of membrane in the lower wall was left exposed. This ensured overlapping and a continuous membrane when applying the membrane in the invert. Thorough cleaning of the invert was necessary in order to facilitate spraying of the membrane in the invert.

Application of the membrane to the invert was achieved by careful spraying. An overlapping horizontal strip of membrane in the lower wall ensured a complete coverage of the tunnel perimeter. Finally the sprayed concrete could be applied in the invert.

The unique features of the sprayed membrane were clearly manifested during the Giswil project. Its versatility in being compatible with other waterproofing systems and tunnel support systems were demonstrated. Furthermore, due to the bond it offered, the opportunity existed for construction of an economical composite single shell lining reduced the risk of uncontrolled water paths typically associated with PVC membranes. A test five years after construction showed that the tunnel was dry and still withstood 4.5 bars of hydrostatic pressure.

7.2 Tunnel de Viret, Lausanne Metro

The new M2 Metropolitan Railway Line was designed to cross Lausanne from South to North. The urban sections were built in 8 tunnels using “cut and cover” construction techniques and the rest as partial face tunneling. Two of these tunnels, including the 275 m long Tunnel de Viret, were waterproofed with Masterseal® 345 spray applied membrane (Fig. 20). The primary lining consists



Figure 20. Masterseal® 345 applied to the entire tunnel contour. A final lining of 30 cm sprayed concrete was applied on top of the membrane (Tunnel de Viret).

of steel arches and mesh followed by a 25 cm thick layer of sprayed concrete.

Originally, sprayed concrete, a traditional PVC sheet membrane and a final lining of cast in-situ concrete were specified for the two tunnels. The alternative solution involved a spray applied waterproofing membrane and an inner lining of fiber-reinforced sprayed concrete (Fig. 21). In terms of construction, this resulted in a considerable amount of time being saved. Furthermore, no shutters were required for the inner shell of concrete. The sprayable membrane alternative proved to be more cost-effective for the project overall compared to a technical solution with sheet membrane.

The primary sprayed concrete lining surface was very rough however no smoothening layer was applied and any large holes were repaired with mortar. Local dripping water was managed via minor polyurethane injection through small hoses fitted in holes drilled in the primary concrete (Fig. 22). Final waterproofing of the tunnel profile was carried out using the Masterseal® 345 spray applied membrane (sprayed to a minimum thickness of 3 mm, Fig. 23).

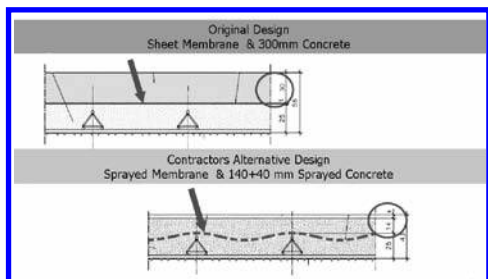


Figure 21. Comparison between original and alternative design (Tunnel de Viret).

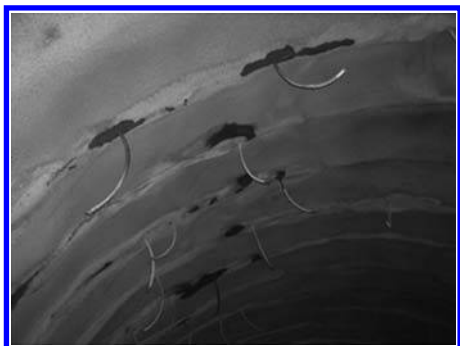


Figure 22. Temporary drainage of water seepages through drillholes and small packers (Tunnel de Viret).



Figure 23. Manual application of Masterseal® 345 (Tunnel de Viret).

A total of 140 mm of sprayed concrete was then applied to the membrane as the inner layer. A 40 mm thick smoothing layer was applied to ensure a good surface finish (Fig. 24). Masterseal® 345 works even on a rough surface with some water but more time is required for surface preparation and wet spot management. Good preparation works were essential. Temporary water drainage is needed for Masterseal® 345 application in the invert. Considerable benefits



Figure 24. Finished tunnel with composite waterproof liner with Masterseal® 345 and sprayed concrete (Tunnel de Viret).



Figure 25. Wet spot management prior to spraying the membrane (Hirtenberger Ammunition Caverns).

in terms of savings on material and time were gained. The construction time was reduced by 3 months.

7.3 Clem Jones Tunnel, Brisbane

The Clem Jones Tunnel is a dual road tunnel providing a direct cross-city link bypassing the CBD. Formerly known as the North-South Bypass Tunnel, the Clem Jones Tunnel (CLEM7) at 6.8 km in length is Brisbane's first major road tunnel. The project features twin two-lane tunnels, including a section up to 60 m below the Brisbane River bed. The main tunnel is a segmentally lined bored tunnel. The contractor was looking for a new, efficient and cost-effective way of waterproofing the cross-passages and intersections with the running tunnel. The tunnel cross passages were waterproofed using a sprayed membrane. The detail at the interface with the segmental lining is illustrated in Figure 6.

7.4 Refurbishment of the Hirtenberger Ammunition Caverns, Austria

In Hirtenberger a cavern system is used to store explosives underground. This facility, which was



Figure 26. Application of a smoothing layer prior to spraying of the Masterseal® 345 waterproofing membrane (Hirtenberger Ammunition Caverns).



Figure 27. Manual spraying of the Masterseal® 345 membrane (Hirtenberger Ammunition Caverns).

about 20 years old, needed a complete refurbishment since the concrete was already partly disintegrated and the caverns could only be used to a limited extent. Since the caverns varied substantially in length, width and height (length 20–58 m, width 5–6 m, height 4–5 m), the designer proposed permanent fiber reinforced sprayed concrete and a fully bonded sprayed membrane for the refurbishment.

After some structural repair was done to the existing lining, a smoothing sprayed concrete layer was applied. Dripping water was channelled and fed into the drainage system by using drainage strips (Fig. 25). Sheet membranes were used for waterproofing of the invert. The Masterseal® 345 sprayed membrane was then applied in the crown and walls (Figures 26 and 27) and connected to the invert's membrane by simple over-spraying. After some curing time a 10–25 cm thick steel fiber reinforced sprayed concrete was placed as the inner lining.

8 CONCLUSION

A wide variety of options exist for tunneling methods and linings. A systematic evaluation of risks can help choose the most appropriate ones. As demonstrated by the recent experiences documented in this paper, an extremely efficient solution for waterproofing has been successfully provided through a fully bonded waterproofing membrane. Where the use of sprayed concrete can be considered as a permanent solution, additional structural benefits can be gained through composite action.

REFERENCES

- BASF, 2007, "Method Statement: Application of Masterseal® 345 Spray Applied Waterproofing Membranes".
- BASF, 2008, "Generic Specification for Masterseal® 345".
- Blümel M., Button E.A., Pötsch M., 2003, Institute for Rock Mechanics and Tunneling, University of Technology Graz, Austria, "Stiffness controlled shear behavior of rock".
- Blümel M., 2005, Institute for Rock Mechanics and Tunneling, Graz University of Technology, Austria, "Laboratory Testing for Soft Rock—a Challenge".
- BMI Institute of Concrete Structures, 2003, Building Materials and Building Physics, "Practical effectiveness of cement-bonded sealing layers for single-shell tunnel construction projects, Masterseal® 345, Laboratory Tests/Practical tests".
- EFNARC Sprayed Concrete Technical Committee, 2007, "EFNARC Nozzleman Certification Scheme".
- Golser, J. & Kienberger, G. 1997, "Permanent sprayed concrete tunnel lining—loading and safety issues".
- Morgan, D.R. 2000, "Shotcrete Guides and Specifications".
- Mott MacDonald, 2009, "Evaluation Report on Masterseal® 345".
- Schulze & Killermann 2000, "Long-term performance of redispersable powders in mortars".
- Swiss Federal Laboratories for Materials Testing and Research, Laboratory for Concrete and Construction Chemistry, EMPA, 2002, "Test of Water-tightness under pressure: Test Report No. 425359.1".
- Technical University of Graz 2008, "Direct Shear Test Results". www.meyco.basf.com

Shotcrete application and optimization at PT Freeport Indonesia's Deep Ore Zone mine

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ABSTRACT: A recent significant increase in production from the Deep Ore Zone (DOZ) Mine of PT Freeport Indonesia requires rapid development and construction. These requirements have resulted in significant challenges in the design and implementation of ground support, including shotcrete implementation. In order to optimize current ground support to anticipate future requirements, some action plans have been completed to review existing shotcrete support performances. These have included: an initial comparison between Steel Fiber Reinforced Shotcrete (FRS) and macro-synthetic FRS, comparative performance tests between welded wire mesh reinforced shotcrete and macro-synthetic FRS in different thicknesses using ASTM C-1550 panels, actual rebound comparison, synthetic fiber impact on mill processes, and quality of shotcrete application improvement. Based on results to date, new ground support systems incorporating synthetic FRS have now been implemented at the DOZ mines and the other future Freeport mines. This paper outlines shotcrete application in the DOZ Block Cave Mine and the application of lessons learned in the management of ground support issues, including synthetic FRS application to ensure safety of workers, work optimization and continued achievement of desired development rates to support the underground mining plan.

1 INTRODUCTION

1.1 Mine location

Freeport Indonesia operates a copper and gold mining complex in the Ertsberg Mining District in the province of Papua, Indonesia (Figure 1). The Ertsberg District is located in the Sudirman Mountains at elevations from 3000 to 4500 metres above sea level. The topography is extremely rugged and rainfall in the mine area averages 5500 mm per year.

Current operations in the district include the Grasberg open pit (180,000 tpd ore) and the DOZ block cave mine (70,000 tpd ore). The DOZ block cave is the third lift of the block cave mine in the East Ertsberg Skarn System (EESS) after the Gunung Bijih Timur (GBT) Mine and the Intermediate Ore Zone (IOZ) Mine (Figure 2). The GBT block cave was in operation from 1980 to 1993 and produced about 60 million tonnes of ore. The IOZ block cave was started in 1994 and had produced over 50 million tonnes of ore when it closed in 2003. The DOZ block cave started production in 2000 and by the end of 2008 it has produced about 115 million tonnes of ore.

1.2 Geotechnical conditions in the DOZ mine

The DOZ block cave mine is situated within the East Ertsberg Skarn System (EESS) which consists



Figure 1. Location of PT Freeport Indonesia mine.

of skarn assemblages locally intruded by variably altered Ertsberg Diorite. The Ertsberg diorite forms the footwall with forsterite skarn, magnetite-forsterite skarn, magnetite and DOZ Breccia (locally known as HALO) and marble in the hanging wall (Coutts et al, 1999). The DOZ Breccia forms a lenticular zone that can be traced continuously across the hanging wall of the eastern half of the DOZ mining block and contains both diorite and skarn fragments within a clay-carbonate matrix. Along the footwall, the diorite was intruded by the skarns producing local alterations.

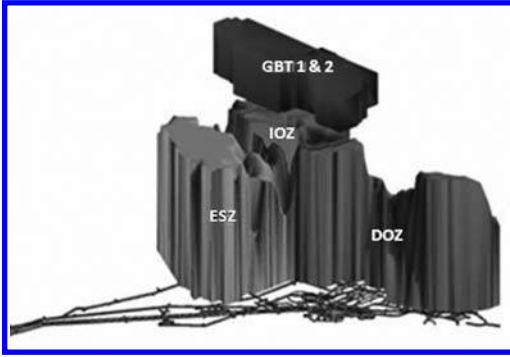


Figure 2. Underground mines complex in the East Erstberg Skarn System in PTFI.

Table 1. Geotechnical classification and its distribution in the DOZ.

Rock type	UCS (MPa)	RQD (%)	RMR class	Percentage (%)
DOZ Breccia	22	45	very poor	10
Marble-Sandstone	53	65	poor	1
Forsterite Skarn	127	84	good	21
Fors-Mag Skarn	57	67	fair	16
Magnetite Skarn	98	71	good	2
Diorite	111	80	good	50

Ground conditions within the EESS system are highly variable. Within zones of good to very good ground conditions there are elongated zones of very poor ground conditions characterized by low strength, low core recovery and low RQD values. The values of the Uniaxial Compressive Strength (UCS), Rock Quality Designation (RQD), Rock Mass Rating (RMR) classification, together with percentage from each rock type, are shown in Table 1.

2 GENERAL SHOTCRETE APPLICATION

2.1 Typical ground support

There are eight (8) general classes of development standard support existing for the DOZ underground:

Class 1A—One Supporting Element—Light Support, Regular Bolting Pattern and Periodical Scaling

Class 1B—One Supporting Element—Light Support, Shotcrete Only

Class 2A—Two Supporting Elements—Medium Light Support, Bolts and Mesh Only

Class 2B—Two Supporting Elements—Medium Light Support, Bolts and Shotcrete Only

Class 3—Three Supporting Elements—Regular Support, Bolts, Welded Mesh and Shotcrete

Class 4—Four Supporting Elements—Heavy Support, Bolts, Welded Mesh, Shotcrete and Threadbars

Class 5—Multiple Supporting Elements—Extra Heavy Support, Bolts, Mesh (Two Layers), Shotcrete (Two Layers) and Threadbars

Class 6—Special Type of Support—Steel sets, Shotcrete Arches

The ground support classes were divided based on some considerations: rock mass condition, long-term requirement or expected drift/tunnel life, frequency of people passing this area, drift or tunnel function, span or dimension, and expected maximum deformation or stress in the field.

2.2 Shotcrete application

Over the last decade Freeport Indonesia has applied wet shotcrete to mechanized underground operations for safety and productivity reasons. In the wet process, the entire mix, including the total amount of water, is fed into the hopper and then pumped to the nozzle. The mix needs to be fluid enough to be pumped, and therefore the introduction of water reducers into the mix is generally essential to maximize strength gain and reduce overall costs.

The nozzle design in wet shotcrete is important because compressed air is added to improve spray velocity; generally, accelerator is added to improve the early strength gain. The main factors to consider when selecting more appropriate shotcrete application process for specific situation are: overall volume required for situation and available time to spray, logistics, overall performance, and overall costs.

2.3 Quality Assurance & Control (QA/QC)

Current QA/QC consists of regular Compressive Strength (CS) and slump tests at the batch plant, field checklists completed by technicians, including preparation of shotcrete in the field, slump tests and early strength after application. Preparation is a key element for successful shotcrete application. The substrate should be free from loose materials, dust, and films, such as oil. This can generally be achieved by using a water jet. Adhesion to weak materials, such as shale and mudstones, is frequently poor, and this factor should be considered



Figure 3. Mesh Installation at DOZ mine.

when designing an appropriate support system. Spraying onto a surface that can vibrate (such as a screen or mesh) can cause problems such as poor in-place density or voids, and can increase rebound as well (Hustrulid and Bullock, 2001). Quality control is important to ensure safe, consistent, and cost effective shotcrete support.

Minimum parameters which are regularly inspected during spraying shifts include: mix design, services (e.g. air volume and pressure of roboshot), strength, and thickness. In general, the inspection and tests will commence at the batch plant, and will be continued at the delivery point, and at a range of times starting 30 minutes after application through to a minimum of 28 days after application. Continuous inspection and testing will lead to a database of shotcrete characteristics and performance through which improvements in shotcrete quality can be determined.

3 SHOTCRETE TESTS

3.1 *Post-crack performance in the laboratory*

To optimize existing ground support and also to determine shotcrete toughness requirements related to existing stress conditions and potential strain burst in the DOZ, UG Geotech and UG QA/QC initiated a project to check the performance of plain and fiber reinforced shotcrete, and the effect of different grades of welded mesh in the shotcrete layer, and also evaluate some combinations of mesh and shotcrete for different thicknesses and positions. This review was triggered by the need to find a more efficient ground support system, as the existing support systems are time consuming to install and involve too many layers. They are also costly in terms of shotcrete thickness, weld mesh utilization, and mesh-time installation. This review was also undertaken in anticipation of the support

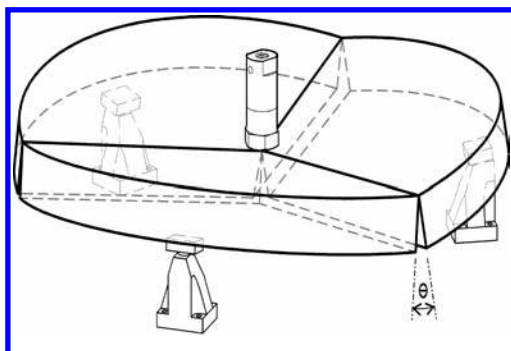


Figure 4. Loading configuration for 1200 mm diameter of ASTM C-1550-like panel subject to a central point load.

requirements for the approaching massive diorite rock mass which is prone to strain burst at ESZ (Erstberg Stockwork Zone—South West of DOZ) and the upcoming deeper mines within the East Erstberg Skarn System (EESS) deposit.

Two tests were conducted by Technologies in Structural Engineering (TSE) Pty Ltd in Sydney, Australia to assess mechanical properties of plain shotcrete with welded wire mesh and FRS with welded mesh, also to investigate various configurations (thickness and number of layers) of FRS and welded mesh. The synthetic fiber used during the tests was the same fiber being used at Freeport, supplied by Elasto Plastic Concrete (EPC) since 2004 (Barchip ‘Shogun’ macro-synthetic fiber).

The performance assessment trials between the different types of ground supports involved two types of panel tests: normal-sized ASTM C-1550 panels of 100 mm thickness, and enlarged C-1550 like panels of 100–200 mm thickness. The normal-sized panels were tested to 40 mm central deflection according to standard practice despite the non-standard 100 mm thickness. The other panel test procedure used in the investigation was an enlarged version of the ASTM C-1550 round panel test in which 1200 mm diameter panels were used instead of the normal 800 mm diameter specimens. Each test involved the development of three radial negative bending hinges (cracks) in response to a central point load while supported on three symmetrically arranged pivots. As the central load point was advanced under displacement control, the specimen cracked and the reinforcement became active. The tests were continued up to a central deflection of about 60 mm, which correspond to an average crack width of 15–20 mm.

The testing machine used for the tests on the large panels was an MTS 244 250 kN servo-hydraulic actuator controlled by an MTS Flextest GT controller operating with a 6000 Hz feedback signal.

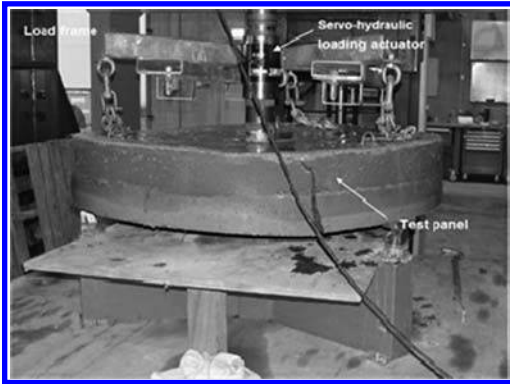


Figure 5. Large 1200 mm diameter panel being tested.

This machine is fully capable of operating in a manner which produces a controlled and constant rate of increase of deflection of the specimen without the intervention of an operator. To limit unstable behavior at first crack, the system stiffness of the test machine, inclusive of the load frame, load cell, and support fixture, must be as great as possible. The test rig used in the present investigation was capable of producing relatively well-controlled post crack behavior in most of the specimens. The central deflection of the panel was taken to be represented by the displacement of the loading piston measured relative to the support mounting of the actuator.

This resulted in the inclusion of extraneous deflections arising from deformation of the load train in the displacement record. However, the test frame was so stiff in the present configuration that extraneous deflections were negligible compared to the deformation of the specimen itself. The displacement imposed on the panel and corresponding load resistance were recorded simultaneously at a rate sufficient to record displacement in increments of no less than 0.05 mm using a digital recording system built into the Flextest GT controller. The data record therefore consisted of at least 1500 points over the 60 mm of displacement. The load point consisted of a steel hemispherical piston with the radius of the hemispherical portion of the head equal to 80 mm, and that of the piston shaft 50 mm.

3.2 Laboratory test result analysis

Each type of ground support had three (3) specimens tested and each test resulted in a load-deflection graph which revealed the post-crack energy absorption and residual load capacity of each specimen. Some of the important results from the laboratory tests are elaborated below.

3.2.1 The effect of shotcrete type (Plain vs FRS)

It was evident that the peak strength of the tested samples was similar for plain shotcrete and FRS when SL62 mesh was included. However, reinforcing fibers substantially improved yielding behavior under high deflections. When used in combination with SL62 mesh, the residual strength and post crack energy absorption were substantially higher for FRS compared to plain shotcrete. The post-crack load capacity of FRS was almost doubled compared to the plain shotcrete for the same displacement or deformation (see Figure 6).

3.2.2 The effect of mesh type on behavior of shotcrete/mesh support

DOZ Mine uses two (2) types of steel mesh: F51 (5 mm diameter wire with an aperture of 100×100 mm) and SL62 (6 mm diameter wire with an aperture of 200×200 mm). Test results showed that both meshes (F51 and SL62) embedded in FRS display similar behavior in pre-peak loading. However in post peak loading, F51 yields more and absorbs more energy. These results not only revealed mechanical properties for different mesh-shotcrete combinations, but also showed the potential to optimize the type of mesh used in the field which will be better in terms of a material management point of view.

3.2.3 The effect of FRS thickness on behavior of ground support

Increasing the overall FRS thickness from 100 to 150 mm substantially increased peak and residual loads. However, for a constant overall thickness (total 150 mm), a FRS combination of an initial 100 mm and secondary 50 mm thick layer was better than a FRS combination of two 75 mm layer thicknesses. This occurred because the layer of steel mesh was placed after the initial thicker layer (farther from the face), thus the mesh was further

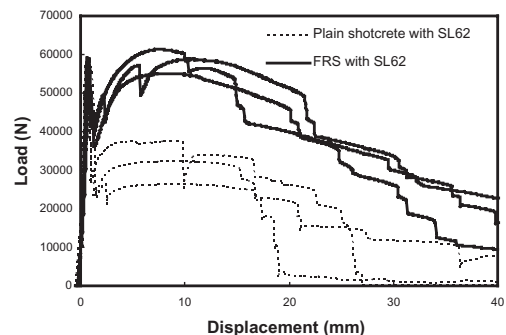


Figure 6. Load-deflection curves for 100 mm thick enlarged plain shotcrete specimens with SL62 mesh compared to 100 mm thick of FRS specimens with SL62 mesh.

from the rock face resulting in improved toughness (see Figure 7).

3.3 Field test observations and evaluation

An initial in-field trial comparison between steel FRS and macro-synthetic FRS was conducted in June–July 2004. During this period the trial was conducted in the DOZ Mine to compare the shotcrete performance, practical application in the field, and the in-place cost as well. At that time the dose of steel fiber used in the shotcrete was 40 kg/m^3 which was compared to different doses of macro-synthetic fiber. The trial results showed that 40 kg/m^3 of steel fiber produced the same toughness level as 6 kg/m^3 of macro-synthetic fiber.

With the current $1,800 \text{ m}^3$ of shotcrete usage per month ($1,200 \text{ m}^3$ for DOZ Mine and 600 m^3 for AB Tunnel development) the total material cost for the same post-crack performance was reduced by approximately 40% through a switch to the macro-synthetic fibers as shown in Table 2. This eliminated steel fibers from further consideration. Current trial results and improved fiber performance showed that the dosage of macro-synthetic fiber could be reduced from 6 kg/m^3 to 5 kg/m^3

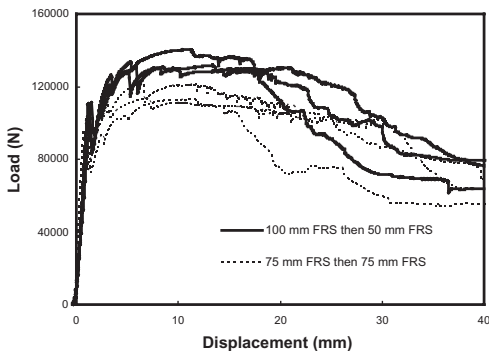


Figure 7. Normal lines showing combination of 100 mm of FRS and 50 mm of FRS compare to the dashed lines which showing combination of FRS 75 mm and 75 mm of shotcrete thickness.

Table 2. Monthly material cost comparison for macro-synthetic and steel Fiber Reinforced Shotcrete.

Type	Price (US\$/kg)	Dose (Kg/m ³)	Volume (m ³ /month)	Cost (US\$)
Plastic	3.97 <i>A</i>	6	1800	42,876 <i>A</i>
Steel	<i>A</i>	40	1800	72,000 <i>A</i>

Note: *A* is multiplier to allow comparison of costs per cubic metre of shotcrete used.

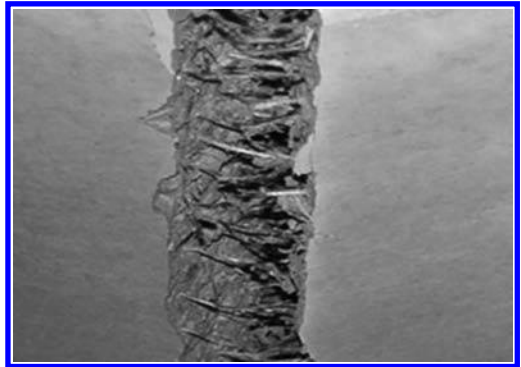


Figure 8. Fiber reinforced shotcrete which is still 'inter-locked' across wider cracks.

without any significant reduction in overall ground support performance in the field.

Currently, two (2) ground support types are being implemented at the DOZ: a) Medium to Poor Ground: F51 mesh followed by 100 mm FRS, another layer of F51 mesh then a final 100 mm FRS; b) Good Ground: F51 mesh followed by 100 mm FRS, another layer of F51 Mesh then a final 50 mm FRS. A cost reduction highlighted in this trial was the possible elimination of one layer combination of mesh and shotcrete for ground support (which resulted in a material and manpower reduction) which in effect will reduce material costs by approximately 30% and associated manpower consumption compared to the previous stabilization procedure. Detailed cost reductions and efficiency increase in overall development cycle is still being reviewed. Positive inputs and feedback were gained from the field crews regarding this change: they reported increased ground support performance (no long-term durability issues due to corrosion as found in the steel FRS), better coverage for open rock discontinuities and cracks, almost no issues related to full remote application in poor ground, and a reduction in scaling requirements for hanging shotcrete.

4 SHOTCRETE CONTRIBUTION TO SAFETY

Safety statistic analysis for rock fall injuries in the DOZ mine for the period 1998–2008 shows that there was a significant production and development rate ramping up from 25,000 tpd to 70,000 tpd, and from 2,000 m lineal development (period 1996) to 16000 m in 2008, with decreasing numbers of rock fall accidents. This excellent outcome with a reducing number of rock fall accidents is a result



Figure 9. Increasing production and development rate with decreasing trend of rock fall injuries in DOZ (1998–2008).

of team-work based on many contributing factors. Some highlights of the contributing factors are as follows: implementation and enforcement of safe mining practices and procedures, introduction of adequate surface support (shotcrete and lower rib support), the increased use of mechanized equipment for drilling and scaling, regular QA/QC inspection, geotechnical monitoring, introduction of an awareness presentation program, and appropriate engineering design in the DOZ mine (Widijanto et al, 2008).

The initiation of the shotcrete program in 2000 using two Viper Roboshots and the enforcement of a safety discipline as part of the Collective Labor Agreement in the year 1999 are examples of the effort to reduce rock fall accidents in the DOZ Mine. One of the items in the Collective Labor Agreement stated that “working under unsupported ground and entering barricaded area such as blasting area, wetmuck area, etc. will cause termination for the employee.”

5 OTHER FIELD CONCERNS

5.1 Actual rebound data

After getting results from the independent shotcrete laboratory, there remained some issues from the field related to shotcrete application with different configurations. Some field data were collected and analyzed to figure out these issues.

Actual shotcrete rebound data was collected from different areas in DOZ to assess the effect of different types of shotcrete application (plain and fiber reinforced shotcrete), different thicknesses of application, and different ground types (poor ground and good ground). From field observations it appeared that the critical factors are operator experience, spraying technique, mixing at the

batch plant, and transportation from batch plant to the field. Figure 10 shows that different configurations of shotcrete design (type and thickness) results in rebound that is still below the maximum rebound allowance.

5.2 Fiber impact at the mill

The other field observation concern is fiber impact on the milling process. Due to rebound during spraying, fallouts, and macro-synthetic fibers floating in water, some fiber flows to existing channels and drainage to the mill-processing area which can disturb mill processing and could impact on mill recovery.

To address this issue, data collection was carried out to identify fiber flowing from different drift elevations and at the same time providing training and refreshment to the shotcrete operator and supervisor about appropriate shotcrete practices and QA/QC. In addition, it was necessary to provide some screens to capture the fiber in the drainage transfer points and to minimize the possible effect in the

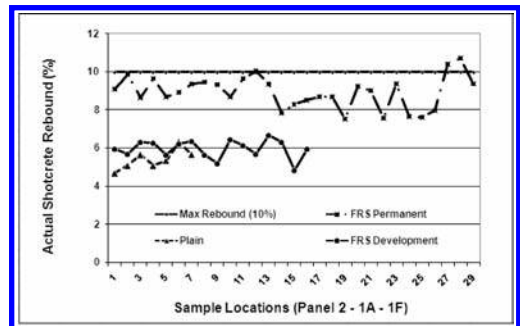


Figure 10. Actual rebound data from different areas, different shotcrete type, and different thickness configuration—still under maximum allowable rebound.



Figure 11. Fiber samples that collected from different drainage transfer points (June, 2008).

mill. These initiatives were conducted in the mine division area and on the other hand, installing vibrating screens on the concentrate thickener feed systems and also self-cleaning (delcor type) screening system on gland/process water tanks led to other significant benefits in reducing the number of fibers flowing to the mill-processing plant.

6 CONCLUSIONS

With a significant ramping up of production and development rates in the DOZ Mine at Freeport Indonesia, there has been a decreasing trend in the number of rock fall accidents. Shotcrete application is one of the contributors to this achievement.

Application of the new ground support types for the DOZ combining 2-layers (150 mm and 200 mm thickness) of macro-synthetic FRS and welded wire mesh (F51) contributed to reduced shotcrete requirements for ground support compared to the previous 3-layer system (250 mm and 275 mm thickness) of ground support. This was accompanied by higher energy absorption capacity and less ground support works. In terms of material cost, there has been an approximately 30–40% cost reduction compared to the previous design, excluding potential savings on mesh layer reduction and increases in efficiency in the overall development cycle. This new design also has proved effective with rebound remaining under maximum allowable limits. It has led to positive feedback from the field crews and no significant impact on the milling process after several measures were implemented in the field.

ACKNOWLEDGMENT

The authors would like to thank the management of PT Freeport Indonesia for permission to publish this paper, also to Elasto Plastic Concrete Pte Ltd for continuous support for the shotcrete trial and improvement in the field. The contribution made by Underground Geotech and QA/QC personnel involved in DOZ Mine to this paper is gratefully acknowledged.

REFERENCES

- American Society for Testing and Materials, 2005. Standard Test Method C-1550. *Flexural Toughness of Fiber Reinforced Concrete (Using Centrally Loaded Round Panel)*. West Conshohocken, PA, USA.
- Aguasa, F. 2008. *Presentation on Shotcrete for PT Freeport Indonesia*, EPC Internal Documentation.
- Archibald, J.F. & Katsabanis, P.T. 2003. *Comparative Use of Shotcrete, Fibercrete and Thin, Spray-on Polymer Liners for Blast Damage Mitigation*; Proc. 105th Annual General Meeting of the Canadian Institute Mining, Metallurgy and Petroleum, Montreal, May 2003.
- Bernard, E.S. 2007. *TSE Report No. 176. Fiber Reinforced Shotcrete Panels Tests*, for Elasto Plastic Concrete P/L. Australia.
- Bernard, E.S. 2008. *TSE Report No. 188. Fiber Reinforced Shotcrete Panels Tests*, for PT Freeport Indonesia. Australia.
- Coutts, B.P., Susanto, H., Belluz, N., Flint, D. & Edw, A. 1999. *Geology of the Deep Ore Zone, East Erstberg Skarn System, Irian Jaya*; Proc. PACRIM 1999. Bali, 10–13 October 1999.
- Hustrulid, W.A. & Bullock, R.L. 2001. *Underground Mining Method*. Society of Mining, Metallurgy, and Exploration Inc: 567.
- Papworth, F. 2002. *Design Guidelines for the use of fiber reinforced shotcrete in ground support*. Shotcrete, Vol. 1: 16–21.
- Widijanto, E. 2008. Freeport Internal Memorandum. *Geotech Review on Shotcrete Support Optimization in DOZ Mine*: 1–5.
- Widijanto, E., Sekeh, P. & Srikant, A. 2008. Freeport Internal Report. *Mitigation of Ground Control Hazards in the DOZ Block Cave-PT Freeport Indonesia*: 1–8.

Air void structures in blended-cement wet-mix shotcrete

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ABSTRACT: The purpose of this experimental investigation was to evaluate the effect of silica fume, Air Entraining Admixtures (AEA), synthetic fibers, powdered polymers, and viscosity admixtures on the air void structure of blended-cement wet-mix shotcretes. This was undertaken because the air void structure of concrete has a crucial effect on durability, especially freeze-thaw resistance. Image analysis was performed on hardened concrete with an automated image analyzer to compare the air void contents and structures of shotcrete mixtures before and after spraying.

The test results for samples without an AEA showed that even though the spacing factors, ranging 450 to 491 μm , decreased a little for all samples after spraying as a result of an increase in the number of air voids, these did not satisfy the usually accepted maximum value of 250 μm . It therefore appears necessary to add an AEA to shotcrete to ensure an appropriate spacing factor and thereby enhance durability. It was then verified through tests that adding an AEA increases the likelihood that the appropriate in-place spacing factor is achieved. Even though the in-place spacing factor increased a little for all samples after spraying, probably as a result of the dissipation of entrained air, these were below the maximum value of 250 μm . The incorporation of silica fume did not affect the in-place spacing factor and specific surface within this study, which may be because the silica fume filled up the very fine voids smaller than 10 μm . The test results for samples with micro-synthetic fibers, powdered polymers, and viscosity admixtures indicated that the in-place spacing factors increased during spraying. This suggested that it is necessary to verify, before use in the field, the air content and in-place spacing factor of shotcrete in those cases where special additives, such as micro-synthetic fibers, powdered polymers or viscosity admixtures are incorporated.

1 INTRODUCTION

Concrete deteriorates under repeated exposure to freezing and thawing. For this reason, an air-entraining admixture (AEA) is frequently used to improve freeze-thaw resistance. The air content is often the easiest measurable quantity to ensure freeze-thaw resistance in the field. There are several techniques available to determine the air content percentage such as the gravimetric (ASTM C 138), volumetric (ASTM C 173), and pressure (ASTM C 231) methods. The total amount of air, however, is not the primary parameter that determines the adequacy of protection against frost damage.

The parameters that define the characteristics of the entrained air-void system, namely the spacing factor and the specific surface, are more appropriate for this purpose. These parameters are defined in ASTM C 457, which is the standard method for microscopic determination of the parameters of air-void systems in hardened concrete. The spacing factor is a useful index of the maximum distance of any point in the hardened cement paste from the periphery of a nearby air void. The calculation of the factor is based on

the assumption that all air voids are equal-sized spheres arranged in a simple cubic lattice as shown in Figure 1 (Neville, 1995). ASTM C 457 provides two procedures for quantification of the spacing factor and the specific surface (that is, the modified point count, and the linear-traverse methods). Both standardized methods are time-consuming to perform and the results are operator-dependant (Pleau et al, 1990).

An image analysis method could be used very easily to determine air content and air-void structure (spacing factor, specific surface, air void size distribution). Moreover, automated image analysis facilitates the process and removes the subjectivity of the operator, which is considered a shortcoming of the

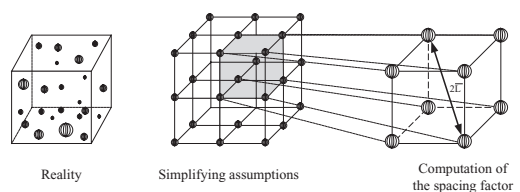


Figure 1. Assumption and computation of spacing factor.

manual operation. Also, research on the air-void structure system of shotcrete, which is changed by dissipation of air during spraying, has never been done though it is very important to durability in a freeze-thaw environment (Jung, 2005).

The objective of this study was to analyse the air void contents and structures of blended-cement wet-mix shotcrete in order to enhance quality control by reducing the variance in the field. To do this, the main experimental variables were selected to be silica fume content (0.0, 4.5, & 9.0% by weight of cement), AEA addition (0.0, 0.005% by volume of concrete), micro-synthetic fiber addition, powdered polymers addition, and viscosity admixture addition. The study was prompted by the results of a recent investigation of strength and durability in high performance shotcrete in Korea (Ma & Kim, 2006). In the work by Ma & Kim, shotcreting was carried out using a site-batched concrete mixture which frequently gave rise to quality control problems in regard to pumpability, shootability, strength development, and durability. The use of pre-blended (or pre-packed) shotcrete materials has therefore been considered as a suitable substitute to solve these problems in the field (Morgan, 1991). Image analysis was performed with an automated image analyzer for comparing the air void contents and structures before and after shotcreting.

2 EXPERIMENTAL PROGRAM

2.1 Shotcrete mix proportions

The aim of the experimental work was to evaluate the effect of silica fume, an air entraining admixture, micro-synthetic fibers, a powdered polymer, and a viscosity admixture on the air void structure of shotcrete. A series of shotcrete mixtures were prepared and sprayed for this investigation. The unit cement content of 440 kg/m³, maximum size of coarse aggregate of 10 mm, fine aggregate ratio $S/a = 0.7$ and water:cement ratio of 0.435 were fixed for all mixtures. The superplasticizer was added as much as required in order to maintain the slump at 120 ± 30 mm. Tables 1 and 2 indicate the experimental variables and details of the shotcrete mix proportions, respectively.

2.2 Materials

Ordinary Portland Cement (OPC) was used in all the shotcretemixtures. The silica fume had a specific gravity of 2.22 and a fineness of 220,000–250,000 cm²/g. The coarse aggregate was a crushed limestone with maximum size of 10 mm; the fine aggregate was natural sand. The specific gravities of coarse

Table 1. Experimental variables considered in the investigation.

Material	Type	Remarks
Cement	OPC (440 kg/m ³)	Fixed
Coarse aggregate	$G_{\max} = 10$ mm	Fixed
Fine aggregate	$S/a = 0.7$	Fixed
Superplasticizer	Polycarboxylate	As required
Silica fume (% bwc)	0.0, 4.5, 9.0	Varied
AE admixture (% v/v)	0.0, 0.005	Varied
Synthetic fiber (% v/v)	0.0, 0.2	Varied
Polymer (% v/v)	0.0, 4.0	Varied
Viscosity admix. (% v/v)	0.0, 0.3	Varied

and fine aggregates were 2.65 and 2.57, respectively. The fine aggregate ratio of the total weight of the aggregates was kept constant at 0.7, and the aggregates were mixed in the surface dry condition. Figure 2 shows the gradations of fine and coarse aggregates with grading limits provided by ASTM C33.

The polymer was added in order to improve the durability of the shotcrete with regard to surface scaling resistance, freeze-thaw resistance and impermeability. A viscosity admixture was used to increase shootability and build-up thickness for repair works. All admixtures were of a powdered type intended for commercially-available cement. The air entraining admixture used in this research was sulfonate silica powder with a light ivory color and 0.9–1.2 bulk density. The polymer was a co-polymer of acetate vinyl and ethylene. The powdered superplasticizer was a polycarboxylate having a brownish color, 370 kg/m³ bulk density, and 6.5 ± 1.0 pH. The viscosity admixture was hydroxypropyl methyl cellulose, having 7.0–12.0% hydroxypropyl content and 28.0–30.0% methoxyl. The micro-synthetic fiber was made of 100% nylon, having a specific gravity of 1.16, 12 mm fiber length, 890 MPa tensile strength, and 5.10 GPa elastic modulus.

2.3 Shotcrete equipment

The wet-mix shotcrete was mixed on site with a pump mixer which had an output of 3.8 m³/hr with 122 bar and 46 hp as shown in Figure 3. The compressed air was supplied from a 390 CFM (11 m³/min) compressor. The pre-blended mixtures were added into the mixer followed by water addition. After they were thoroughly mixed, the concrete mixture was dumped from the mixer into the pump hopper. The concrete was then pumped using a twin-piston hydraulic pump mounted with a rocker valve.

Table 2. Mix proportions of shotcrete.

Mix. ID	Quantity (kg/m ³)										Compressive strength (MPa)
	Water	Cement	Sand	Gravel	Silica fume	AEA	Super-plasticizer	Fiber	Polymer	Viscosity Ad.	
SF0	191	440	1122	496	0 (0%)	—	1.012 (0.23%)	—	—	—	45.2
SF4.5	191	420	1118	494	20 (4.5%)	—	1.012 (0.23%)	—	—	—	51.5
SF9	191	400	1113	492	40 (9%)	—	1.188 (0.27%)	—	—	—	50.3
SF0A	191	440	1122	496	0 (0%)	0.022 (0.005%)	0.748 (0.17%)	—	—	—	47.5
SF4.5A	191	420	1118	494	20 (4.5%)	0.022 (0.005%)	0.836 (0.19%)	—	—	—	55.7
SF9A	191	400	1113	492	40 (9%)	0.022 (0.005%)	1.056 (0.24%)	—	—	—	57.6
SF9A-F	191	400	1113	492	40 (9%)	0.022 (0.005%)	1.056 (0.24%)	0.880 (0.2%)	—	—	51.3
SF9A-FP	191	400	1097	485	40 (9%)	0.022 (0.005%)	0.528 (0.12%)	0.880 (0.2%)	17.6 (4%)	—	42.9
SF9A -FPV	191	400	1110	485	40 (9%)	0.022 (0.005%)	0.704 (0.16%)	0.880 (0.2%)	17.6 (4%)	1.320 (0.3%)	39.9

Note:

SFX: X indicates silica fume-cement ratio by weight;

SFXA: A indicates incorporation of air entraining admixture;

SF9A-F indicates a mixture of 9% silica fume, with AE admixture and fiber;

SF9A-FP indicates a mixture of 9% silica fume, with AE admixture, fiber and polymer;

SF9A-FPV indicates a mixture of 9% silica fume, with AE admixture, fiber, polymer and viscosity admixture.

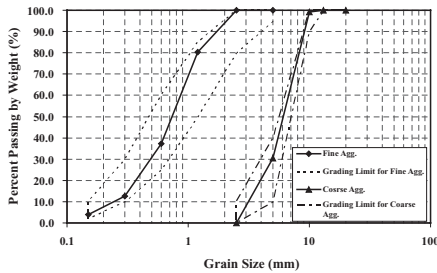


Figure 2. Particle size distribution curve for aggregates.



Figure 3. Shotcrete pump with mixer.

2.4 Sampling

The samples for the tests were taken at two points in the production process: before pumping and after shotcreting. Once the concrete was thoroughly mixed in the pump mixer, the concrete mixture was used to fill cylinders measuring 150 × 300 mm according to ASTM C39. The cylinder samples were moved to a curing room just after placing and cured at 20°C and 100% RH until slicing for image analysis.

After shotcreting into a form measuring 840 × 840 mm with 45° sides (as shown in Figure 4) the concrete was allowed to cure in the curing room. Core samples were taken measuring 100 mm in diameter after 28 days. Slices were cut and used for the image analysis of air content and air void structure of samples after shotcreting.

2.5 Air void analysis

The concrete cylindrical specimens were cut with a water-cooled diamond saw to produce 50 mm thick samples. One face of the sample was polished on a water-cooled rotating lap followed by 60, 100, 220, 320, 400 and 600 grit SiC papers as shown in Figure 5.

Image analysis was performed on the concrete specimens with a HF-MA C01 machine, as shown

in Figure 6, to obtain air-void information such as total area, total area of voids, total number of air voids, average area of an air void, average diameter of an air void, and air contents. These experimental data were used in calculating the spacing factor and specific surface according to ASTM C 457.

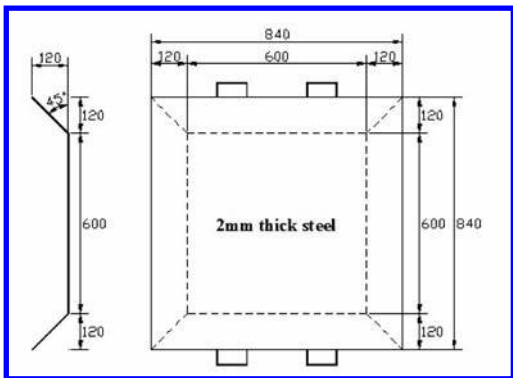


Figure 4. Shotcrete panel for sampling (units in mm).

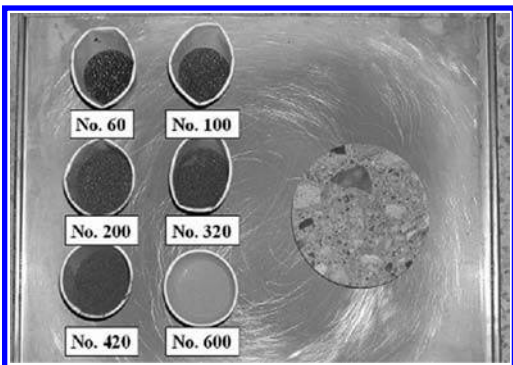


Figure 5. Polishing a specimen for image analysis.



Figure 6. Image analysis set-up of HF-MA C01 for air-void analysis.

This machine could classify air voids from 10 μm to 6000 μm into 23 classes and obtain air void size distributions and air contents. Air voids bigger than 1,000 μm were considered entrapped air voids, thus only the voids between 10 and 1,000 μm were considered entrained air.

3 EXPERIMENTAL RESULTS AND DISCUSSION

3.1 Air void structure without AE admixture

The air content and number of air voids for the samples without AE admixture were analyzed, before and after spraying, for varying amounts of silica fume. Figure 7 compares the air content with the air void size distribution before shotcreting (BS) and after shotcreting (AS), indicating that the air voids larger than 300 μm dissipated during spraying. This would have been caused by expulsion of the bigger air bubbles during shooting. The air voids between 10 μm and 100 μm diameter, which play a crucial role in freeze-thaw resistance, remained unchanged or experienced a small increase in number after spraying.

Figure 8 compares the air contents in three ranges: between 10 and 100 μm , between 100 and 1,000 μm , and between 1,000 and 6,000 μm . The air content between 10 μm and 100 μm stayed unchanged, while those bigger than 100 μm decreased quite a lot after shooting. This also indicates that shotcreting retained the smaller entrained air voids while dissipating the larger entrapped air voids in the mix without AEA. The air void size distribution was quite similar for all samples.

Figures 9 and 10 compare the number of air voids with air void size distribution in three groups, respectively, before and after shotcreting. These indicate that the number of air voids smaller than 100 μm increased after shotcreting, while those larger than 100 μm decreased.

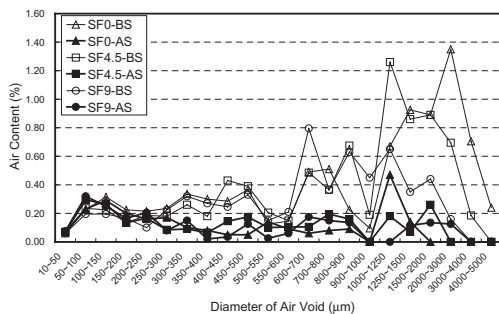


Figure 7. Distribution of air content without AEA.

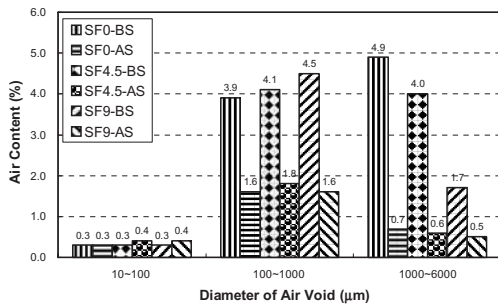


Figure 8. Comparison of air content without AEA.

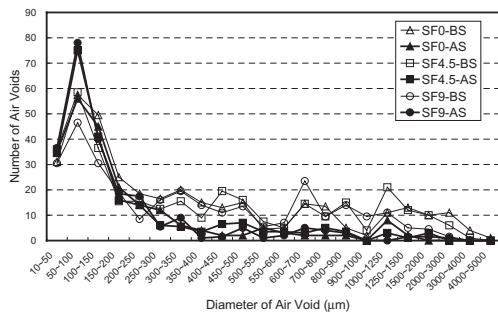


Figure 9. Distribution of air void number without AEA.

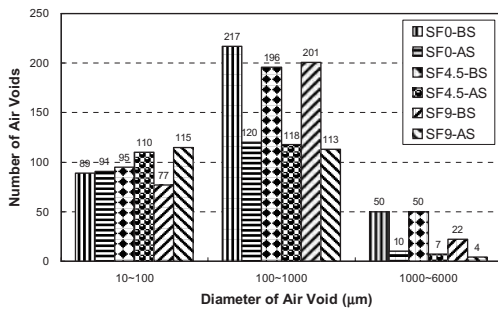


Figure 10. Comparison of air void number without AEA.

Table 3. Summary of air void structure of hardened shotcrete without AEA.

Mix. designation	Air content (%)	Spacing factor (μm)	Specific surface (mm ² /mm ³)
SF0-B*	9.1	450	8.28
SF0-A**	2.6	398	17.72
SF4.5-B	8.4	479	8.33
SF4.5-A	2.8	403	17.29
SF9-B	6.5	491	9.63
SF9-A	2.5	392	20.46

* BS: Before Shotcreting, ** AS: After Shotcreting.

Table 3 summarizes the test results for samples without AE admixture in terms of air content, spacing factor and specific surface. The in-place spacing factors decreased for all samples after shotcreting as a result of an increase in the number of small air voids. The specific surface, also, increased for all samples after shotcreting as a result of an increase in the number of small air voids.

The incorporation of silica fume did not affect the in-place spacing factor and specific surface within this study, which may be because the silica fume filled up the very fine voids smaller than 10 μm. The in-place spacing factors for samples ranged from 392 to 403 μm, which does not satisfy the appropriate range required for freeze-thaw resistance (200~250 μm) proposed by Powers (1949) and used today in many shotcrete specifications. Thus, it is necessary to add an AE admixture in order to increase the likelihood that there is an appropriate in-place spacing factor and durability for the in-place shotcrete.

3.2 Air void structure with AE admixture

Figures 11 to 14 compare the variation of air void structure for samples mixed with 0.005% v/v AEA, before and after shotcreting. Figure 11 compares the air contents with air void size distribution before and after shotcreting, indicating that the air voids dissipate for all air void sizes. The air content dissipates the most for air voids between 100 μm and 1,000 μm size as shown in Figure 12. The air contents of samples with AE admixture decreased from 16.4% to 12.8% as the quantity of silica fume increased from zero to 9%. This may be due to the filling effect of silica fume in shotcrete paste caused the pozzolanic reaction, together with adsorption of AE admixture onto the non-carbonated particles of silica fume. Figure 13 compares the number of air voids for samples with AE admixture before and after shotcreting. The number of air voids decreased

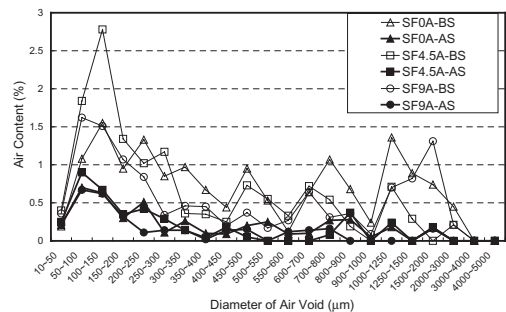


Figure 11. Distribution of air content with AEA.

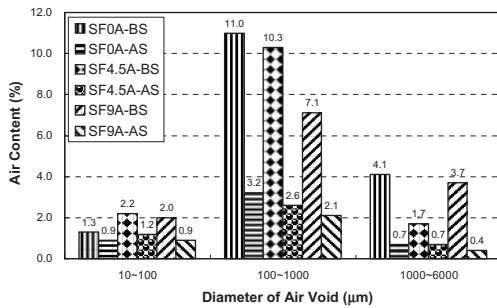


Figure 12. Comparison of air content with AEA.

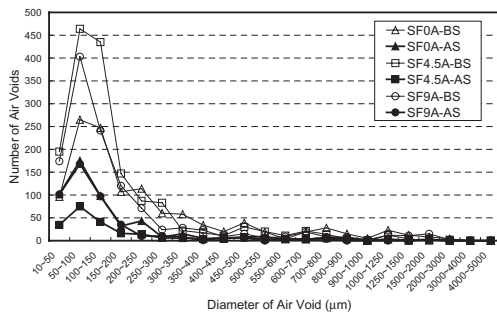


Figure 13. Distribution of air void number with AEA.

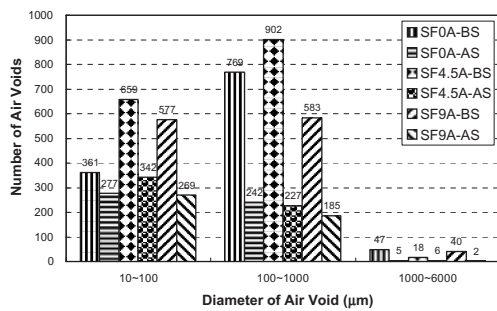


Figure 14. Comparison of air void number with AEA.

Table 4. Summary of air void structure of hardened shotcrete with AEA.

Mix. designation	Air content (%)	Spacing factor (μm)	Specific surface (mm ² /mm ³)
SF0 A-BS*	16.4	136	14.92
SF0 A-AS**	4.8	236	22.66
SF4.5 A-BS	14.2	101	23.16
SF4.5 A-AS	4.5	208	26.49
SF9 A-BS	12.8	135	19.43
SF9 A-AS	3.4	225	28.00

* BS: Before Shotcreting, ** AS: After Shotcreting.

for entrained air but remained for entrapped air after shotcreting. The air void number between 10 and 100 μm is greatest for 4.5% silica fume both before and after shotcreting.

Table 4 summarizes the test results of samples with AE admixture in terms of air content, spacing factor and specific surface. The in-place spacing factors for samples after shotcreting ranged from 208 to 236 μm, which satisfies the appropriate range (200~250 μm) proposed by Powers (1949). The in-place spacing factors increased for all samples after shotcreting as a result of dissipation of entrained air. The in-place specific surface also increased for all samples after shotcreting. It was therefore verified that adding an AE admixture assisted the achievement of an appropriate in-place spacing factor.

3.3 Air void structure with fiber, polymer, and viscosity admixture additions

The purpose of this series of tests was to investigate the effect of micro-synthetic fibers, powdered polymers and viscosity admixtures, which are very frequently used for repair works, on the air void structure of shotcrete. The control sample was a mixture containing 9% silica fume and 0.005% AE admixture.

The results are shown in Figures 15 to 18. The quantity and number of air voids decreased for all samples in a similar pattern after shotcreting. However, the sample with the viscosity admixture showed a somewhat different pattern, indicating that the decrease after spraying is small for air voids between 10 and 100 μm and between 1,000 and 6,000 μm. The air content of samples with polymer showed a high value before shotcreting and this led to a 1% larger air content compared to the control after shotcreting.

The decrease in the air void size between 10 and 100 μm and between 1,000 and 6,000 μm was smaller for the samples with the viscosity admixture

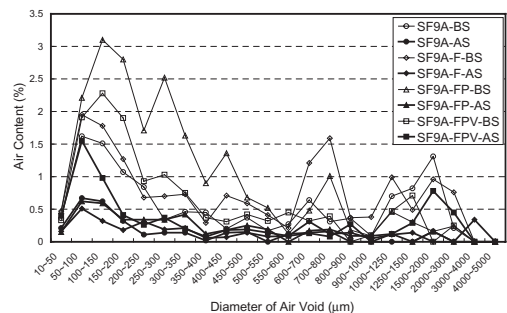


Figure 15. Distribution of air content with fiber, polymer & viscosity admixture.

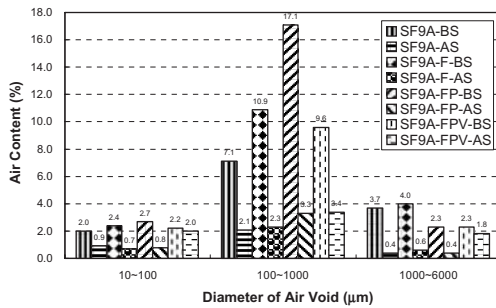


Figure 16. Comparison of air content with fiber, polymer & viscosity admixture.

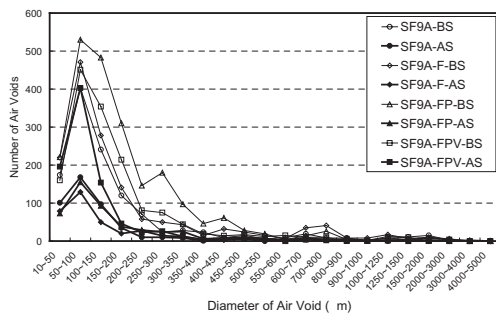


Figure 17. Distribution of air void number with fiber, polymer & viscosity admixture.

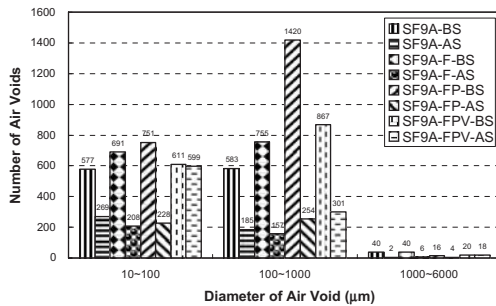


Figure 18. Comparison of air void number with fiber, polymer & viscosity admixture.

and micro-synthetic fibers. Table 5 summarizes the test results for samples with micro-synthetic fibers, powdered polymer, and the viscosity admixture. The in-place spacing factors for the samples with fiber increased from 109 to 289 μm after spraying, which resulted from the decrease in the number of air voids between 10 and 1,000 μm . The in-place spacing factors for the samples with polymer and viscosity admixture increased from 76 to 289 μm and 111 to 171 μm , respectively, after spraying.

Table 5. Summary of air void structure of hardened shotcrete with addition of fiber, polymer & viscosity admixtures.

Mix. designation	Air content (%)	Spacing factor (μm)	Specific surface (mm^2/mm^3)
SF9 A-BS*	12.8	135	19.43
SF9 A-AS**	3.4	225	28.00
SF9 A-F-BS	17.3	109	17.85
SF9 A-F-AS	3.6	289	21.27
SF9 A-FP-BS	22.1	76	20.61
SF9 A-FP-AS	4.5	251	22.54
SF9 A-FPV-BS	14.1	111	22.19
SF9 A-FPV-AS	7.2	172	26.49

* BS: Before Shotcreting, ** AS: After Shotcreting.

The in-place spacing factors increased for all samples after shotcreting as a result of dissipation of entrained air. The in-place specific surfaces also increased for all samples after shotcreting. The in-place spacing factors for samples with micro-synthetic fibers, powdered polymer, and viscosity admixture ranged from 172 to 289 μm , which is close to the limit of 250 μm proposed by Powers (1949). Thus, it is necessary to verify the air content and spacing factor for samples with special additives, such as micro-synthetic fibers, powdered polymers, or viscosity admixtures, before using them in the field.

4 CONCLUSIONS

The purpose of this experimental work was to evaluate the effect of silica fume, air entraining admixture, micro-synthetic fiber, powdered polymer, and viscosity admixtures on the air void structure of blended-cement wet-mix shotcretes. Image analysis was performed on hardened concrete with an automated image analyzer for comparing the air void content and structure before and after spraying.

The test results for samples without an AEA showed that even though the spacing factors, ranging 450 to 491 μm , decreased a little for all samples after spraying as a result of an increase in the number of air voids, these did not satisfy the usually accepted maximum value of 250 μm . It therefore appears necessary to add an AEA to shotcrete to ensure an appropriate spacing factor and thereby enhance durability.

It was then verified through tests that adding an AEA increases the likelihood that the appropriate in-place spacing factor is achieved. Even though the in-place spacing factor increased a little for all samples after spraying, probably as a result of the

dissipation of entrained air, these were below the maximum value of 250 μm . The incorporation of silica fume did not affect the in-place spacing factor and specific surface within this study. The test results for samples with micro-synthetic fibers, powdered polymers, and viscosity admixtures indicated that the in-place spacing factors increased during spraying. This suggested that it is necessary to verify, before use in the field, the air content and in-place spacing factor of shotcrete in those special cases where special additives, such as micro-synthetic fibers, powdered polymers or viscosity admixtures are incorporated.

REFERENCES

- ACI-506R-05, 2005. *Guide to Shotcrete* ACI Committee 506 ACI International.
- ASTM C 138/C 138M-01a, 2001. Standard Test Method for Density (Unit Weight), Yield, and Air Content (Gravimetric) of Concrete, ASTM International, West Conshohocken, PA.
- ASTM C 173/C 173M-01, 2001. Standard Test Method for Air Content of Freshly Mixed Concrete by the volumetric Method, ASTM International, West Conshohocken, PA.
- ASTM C 231-91, 1997. Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method. ASTM International, West Conshohocken, PA.
- ASTM C 457-98, 1998. "Standard Test Method for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete," ASTM International, West Conshohocken, PA.
- Jung, W.K. 2005. Development of Plane Spacing Factor for Evaluation of Freezing-Thawing Durability in Concrete, Ph.D. Thesis. Department of Civil Engineering, Graduate School, Kangwon National University.
- Ma, S.J. & Kim, D.M. 2006. "A Study on Field Test of High-Strength Shotcrete using High-quality Additions and Accelerators", *Journal of the Korean Society of Civil Engineers*, Vol. 26, No. 2C, pp. 121–131.
- Morgan, D.R. 1991. "High Early Strength Blended-Cement Wet-Mix Shotcrete", *Concrete International*, No. 5, May 1991, pp. 35–39.
- Neville, A.M. 1995. *Properties of Concrete*, Fourth Edition, Longman, pp. 546–547.
- Pleau, R. Plante, P. Gagne, R. & Pigeon, M. 1990. "Practical Consideration Pertaining to the Microscopical Determination of Air-Void Characteristics of Hardened Concrete (ASTM C 457)," *Cement, Concrete, and Aggregates*, CCAGDP, Vol. 12, No. 2, pp. 3–11.
- Power, T.C. 1949. "The Air Requirement of Frost-Resistant Concrete". Research Laboratories of the Portland Cement Association, Vol. 29.